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INCLUDING PRESTRESSED CONCRETE

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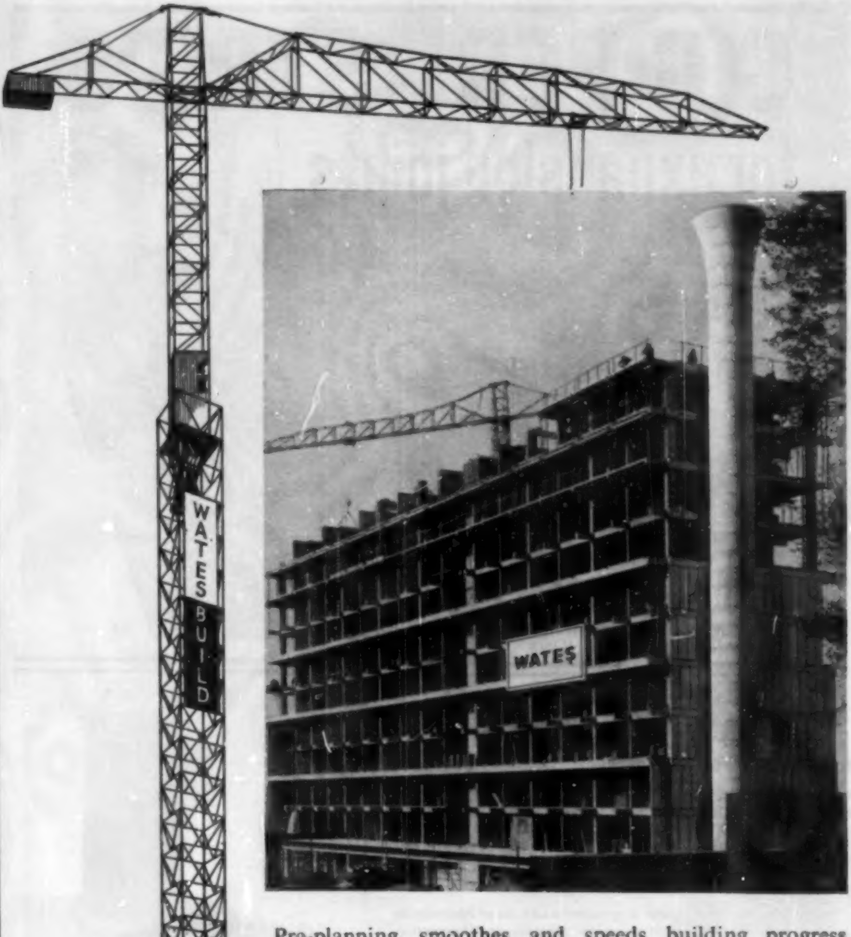
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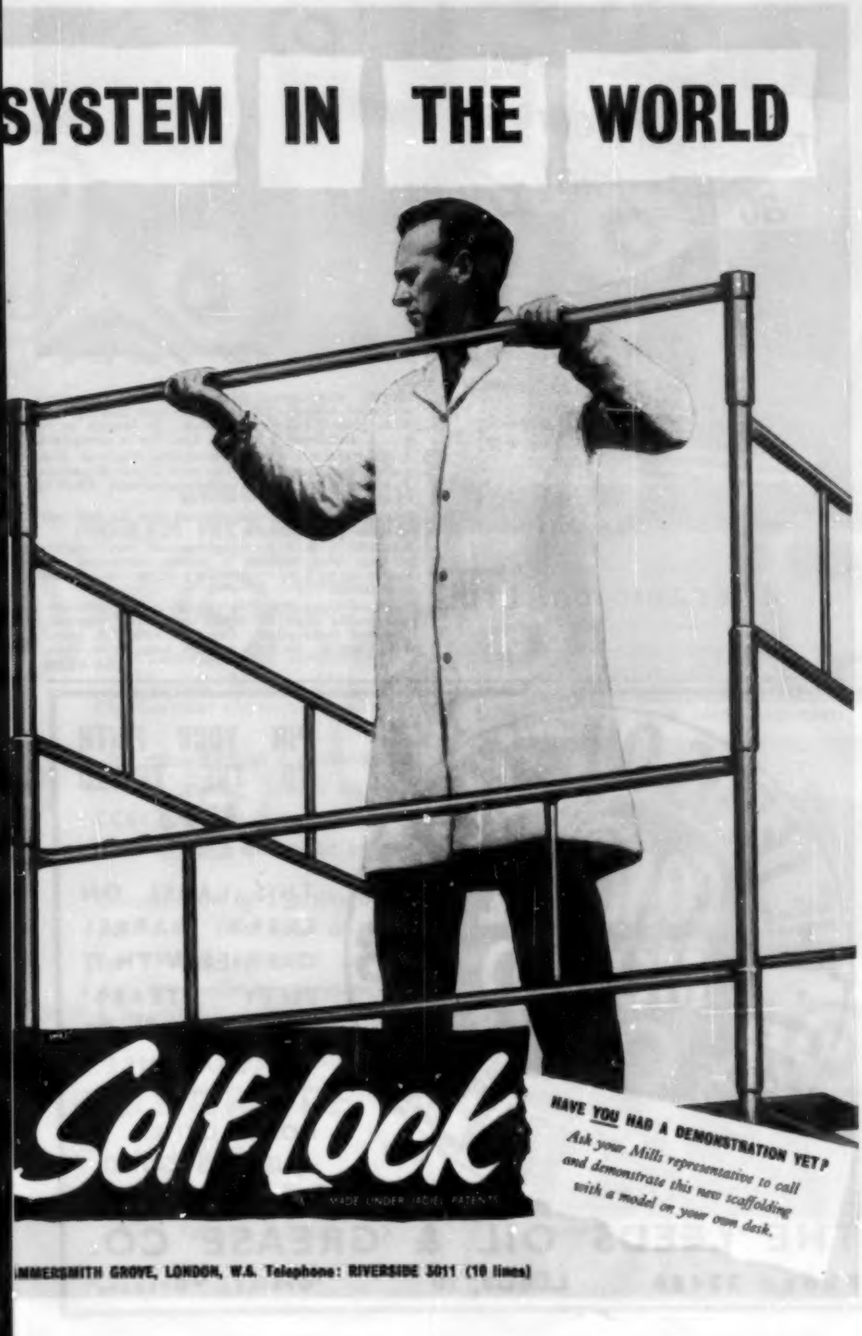
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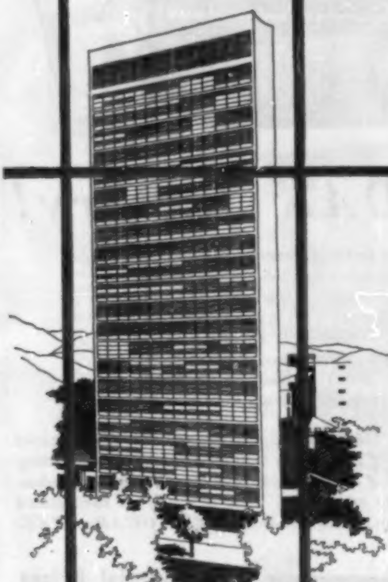

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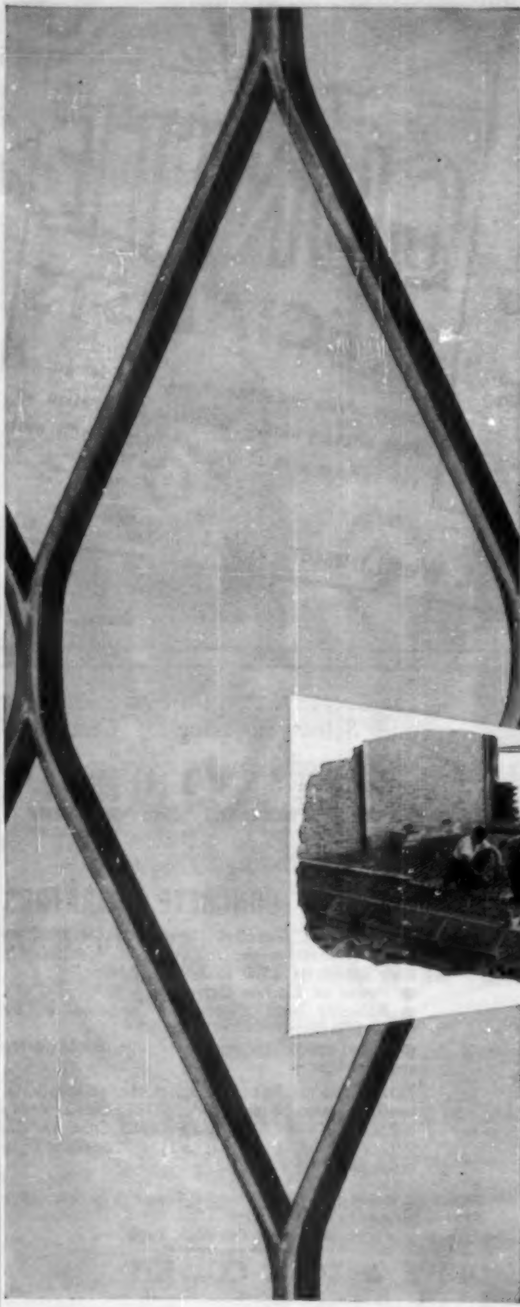

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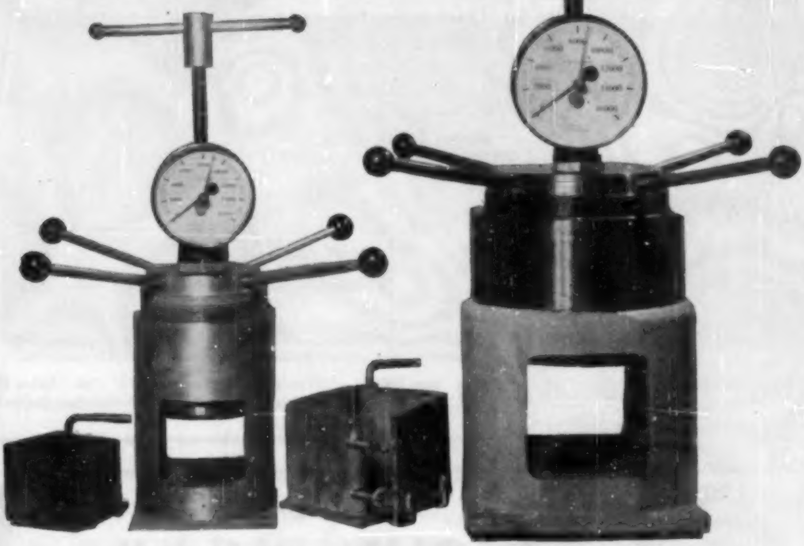

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

Volume LI, No. 12. LONDON, DECEMBER, 1956.

EDITORIAL NOTES

The Application of the Results of Tests.

THE testing of a few samples as a means of assessing the properties of a large number of the same product inevitably results in the acceptance of assumptions as truths. If we were to believe that every batch of concrete throughout a structure would have the same strength as that of small samples tested before the work started, or that all the reinforcement would be the same as the short pieces tested, we would assume that probability was certainty. We can predict that the sun will rise to-morrow because it has been known to do so regularly for so long that probability becomes certainty. In the case of other natural phenomena, such as the force of gales, the height of waves, degrees of heat and cold, and so on, records have been kept for a few years only; in such cases it is safe to assume that the possibility that the greatest recorded winds and waves and heat and cold will be exceeded is in fact a certainty. These are, of course, the reasons why factors of safety are used as precautions against unknown natural forces, unknown human fallibility, and unknown variations in processes that may be always the same in theory but are known to vary in practice.

According to our knowledge of the probability of differences in the properties of what are supposed to be the same products, or of the variations in mechanical processes, so the nature of tests and the use made of the results vary, or should vary, if they are to give a reasonably reliable guide to the properties of the remainder of the products or to the continuing uniformity of a process. There are, however, many differences, some of them not easily explained, in the numbers of samples tested and of using the results in the concrete industry. For example, British Standard Code of Practice No. 114 requires that in making his calculations an engineer may assume that the compressive strength of concrete will not be less than a certain value provided that the average strength of three cubes (the strongest and weakest of which do not vary by more than 15 per cent. from the average) made before the work starts exceeds the required strength in the work by about 15 per cent. This applies to all grades of concrete, including high-grade concretes. On the other hand, the specification of the Ministry of Works for "Quality Controlled Concrete" requires that twelve preliminary test cubes must all exceed the required strength by 15 per cent. The average of three cubes

and every one of twelve cubes are very different bases on which to assess the properties of concrete, and suggest that the Ministry of Works is much more sceptical of the probability of preliminary tests accurately predicting the strength of concrete made with the same materials and plant when placed in the work, even when supervision is very strict. The U.S.A. standard is also more stringent than the British, and requires that the average strength of four or more specimens be used as the criterion of the strength of the concrete in the work.

In the case of precast concrete products the tests differ according to the product. This is to be expected, for the strengths required in different products vary and the seriousness of the consequences of failure also varies. There are, however, some curiosities in the revised British Standards for precast products issued this year. In the standard for precast lintels the customer may select for a breaking-load test one lintel per hundred ordered, and this is the only acceptance test recognised. Porosity is ignored—the standard merely states that the maker should (not must) satisfy himself that the concrete is suitable for the purpose if the lintels are to be used in exposed positions, and there is no provision by which the customer can refuse to accept them if in his opinion the lintels are too porous for their purpose. The compressive strength required is a minimum of 2500 lb. per square inch at twenty-eight days. Standard designs for lintels for external use are given, including one 6 ft. $5\frac{3}{16}$ in. long by $4\frac{3}{16}$ in. wide with two $\frac{1}{2}$-in. bars near the bottom, and this is coupled with a clause that the cover of concrete need be only $\frac{3}{4}$ in. Whether the bars are spaced to give as much cover as possible within this width, or whether the specified minimum cover is used, this is a surprisingly thin protection of such weak concrete, and one that few engineers would accept. It seems that the compilers of this standard are much too optimistic in their assessment of the probable durability of exposed lintels made to meet their requirements, and assume far too much in considering that its breaking strength at twenty-eight days is the only property considered to be important in a lintel.

The revision of the standard for paving flags requires that three flags be tested for orders up to 1000 sq. yd. (say, one flag in five hundred). If any of these samples fail to pass the test for transverse strength then three more may be tested, and if these pass the test then all the flags must be accepted. If it should happen that all three of the flags fail in the first test and all pass in the second, then the customer is expected to believe that all the flags are in accordance with the standard although it has been proved that half of the specimens tested have failed. In the case of the revised standard for concrete roofing tiles there is another variation in that the averages of test results are used. For example, in the permeability test three tiles may be tested out of ten thousand, and if their average impermeability meets the requirement they must all be accepted. In the absence of any limit of deviation, it could happen that two of the tiles were sub-standard and one sufficiently above the standard to enable the average to comply with it. In this case it would be assumed that ten thousand tiles complied with the standard, although tests showed the probability that two out of every three would not do so.

Engineers and contractors would be well advised to study British Standards carefully before specifying precast concrete units to comply with them, for the tests relate to products already made and perhaps delivered.

The Analysis of Frames with Non-prismatic Members.

THE COLUMN-ANALOGY METHOD.

By E. S. E. LYLE, B.Sc., A.M.I.C.E.

In this journal for June 1956 the writer described the use of the column-analogy method for the analysis of gable frames consisting of straight prismatic members. In the following the method is extended to the analysis of any type of single-bay frame of one story containing straight, curved, or non-prismatic members. The same notation is used.

The analogous areas and other properties of any straight prismatic members may be found by the method previously described. The properties of other members are found by finite integration. The member is divided along its axis into a convenient number of short sections, preferably, but not necessarily, equal in length. The lengths of the sections should be such that I does not change greatly or suddenly within the section, and i_x, etc., for the section should be so small in relation to I_x, etc., for the whole frame that they can be neglected. Each section may be considered as a short straight prismatic member with a moment of inertia equal to that at its mid-point. The frame may now be analysed in the usual way, the only difference being the increase in the amount of arithmetic due to the increased number of sections.

Example I.—In a hinged frame with tapering columns (*Fig.* 1) it is assumed that the width of all the members is the same and that I may therefore be assumed to be proportional to d^3 where d is the depth at the point considered. If the width is not constant, I should be based on the actual cross sections. The member BC is dealt with as described in the previous article, and AB and CD are divided into six sections of equal length. The evaluation of I_y is shown in *Table* I, which shows that I changes quickest within section 1. At A it is 1, and at the top of the section it is 1·652. The width $1/I$ of the analogous column is 1 at A, at the mid-point of the section it is 0·771, and at the top 0·605. The value 0·771 is used in the calculations. A more accurate value would be the average (0·781), but the error is small, particularly as the section concerned lies close to the X–X axis and therefore has only a small influence on the final value

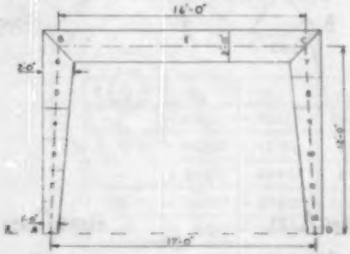


Fig. 1.

Table I.

Section	$1/I$	L	a	y	ay^2
1	0·771	2·0	1·542	+ 1·0	1·5
2	0·484	2·0	0·968	+ 3·0	8·7
3	0·325	2·0	0·650	+ 5·0	16·3
4	0·228	2·0	0·456	+ 7·0	22·4
5	0·167	2·0	0·334	+ 9·0	27·1
6	0·125	2·0	0·250	+ 11·0	30·3
BE	0·125	8·0	1·00	+ 12·0	144·0

$$\begin{array}{r} 250·3 \\ 2 \\ \hline I_y = 501 \end{array}$$

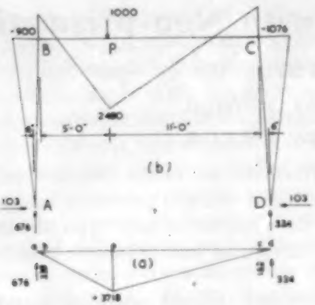


Fig. 2.

TABLE II.

Section	a	m̄	p=m̄a	y	py
1	1·542	+ 28	+ 43	+ 1·0	+ 43
2	0·968	+ 84	+ 81	+ 3·0	+ 243
3	0·650	+ 141	+ 92	+ 5·0	+ 460
4	0·456	+ 197	+ 90	+ 7·0	+ 630
5	0·334	+ 253	+ 85	+ 9·0	+ 765
6	0·250	+ 309	+ 77	+ 11·0	+ 847
B P	0·625	+2028	+ 1270	+ 12·0	+15240
P C	1·375	+1940	+ 2670	+ 12·0	+32040
7	0·250	+ 148	+ 37	+ 11·0	+ 407
8	0·334	+ 121	+ 40	+ 9·0	+ 360
9	0·456	+ 94	+ 43	+ 7·0	+ 301
10	0·650	+ 67	+ 44	+ 5·0	+ 220
11	0·968	+ 41	+ 40	+ 3·0	+ 120
12	1·542	+ 13	+ 20	+ 1·0	+ 20

$M_y = +51696$

Point	m_s	m_i	m
B	+ 338	+1238	-900
P	+ 3718	+1238	+2480
C	+ 162	+1238	-1076

$m = \frac{51696}{501} y = 103{\cdot}2\,y.$ H = 103.

$V_a = 676.$ $V_D = 324.$

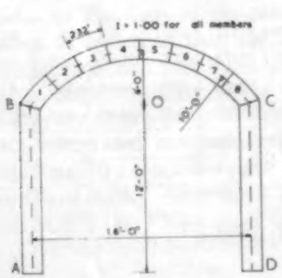


Fig. 3.

TABLE III.

Origin at O

Section	I_1	L	a	y	ay
AB	1·0	12·0	12·00	- 6·00	- 72·0
1	1·0	2·32	2·32	+ 0·86	+ 2·0
2	1·0	2·32	2·32	+ 2·36	+ 5·5
3	1·0	2·32	2·32	+ 3·40	+ 7·9
4	1·0	2·32	2·32	+ 3·93	+ 9·1
			21·28		- 47·5

$\bar{y} = \frac{-47{\cdot}5}{21{\cdot}28} = -2{\cdot}23$

TABLE IV.

Section	a	x	y	ax^2	ay^2	i_y
AB	12·0	- 8·00	- 3·77	76·8	170·6	144
1	2·32	- 7·24	+ 3·09	121·6	22·2	–
2	2·32	- 5·48	+ 4·59	69·7	48·9	–
3	2·32	- 3·41	+ 5·63	26·9	73·6	–
4	2·32	- 1·16	+ 6·16	3·1	88·2	–
				989·3	547·5	

A = 4256. I_x = 1979. I_y = 1095.

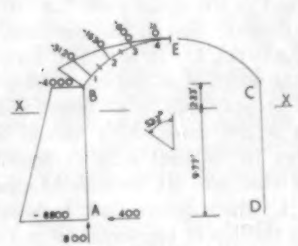


Fig. 4.

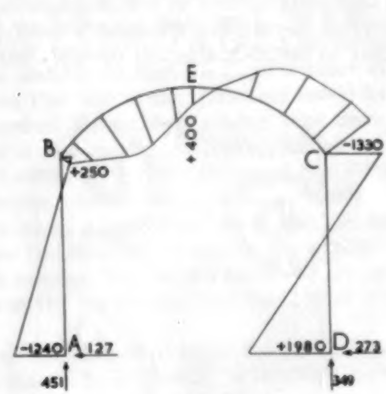


Fig. 5.

of I_y. The value of i_y for section 1 is $\frac{1}{12} \times 0{\cdot}771 \times 2^3 = 0{\cdot}257$ and for section 6 is 0·042; these are small enough to be neglected and the choice of six sections for each column is therefore reasonable. If twelve sections were used I_y would have been 512, that is a difference of about 2 per cent.

The frame is analysed in *Fig.* 2 and *Table* II for a vertical concentrated load. The static bending-moment diagram chosen (*Fig.* 2a) is that for a beam simply supported at A and D. The final moments are shown in *Fig.* 2b. If twelve sections were analysed, m_i would have been $101{\cdot}4y$ instead of $103{\cdot}2y$, that is a difference of less than 2 per cent. It is seen, therefore, that the work involved in using a larger number of sections is not justified unless the frame is particularly important and the anticipated loads can be estimated very accurately, which is not generally possible.

EXAMPLE II.—A frame rigidly fixed at A and D and with a top member curved to a radius of 10 ft. is shown in *Fig.* 3. The elastic centre, which must lie on the Y–Y axis, is calculated in *Table* III and the properties of the analogous column in *Table* IV. In each case, due to its symmetry, only half the frame need be considered. The frame is analysed for an inwardly-acting radial load of 100 lb. per foot. The static bending-moment diagram chosen is that for the cantilever ABE (*Fig.* 4). The bending-moment at any point on the curved member at an angle α from the vertical through the centre of the arc is

$$m = wr^2(1 - \cos\alpha) = 100 \times 10^2(1 - \cos\alpha).$$

TABLE V.

Section	a	$\bar{m}$	p	x	y	px	py
A B	12·0	−4000	−48000	−8·00	−3·77	+384000	+181200
A B		−2400	−28800		−5·77	+230400	+166200
1	2·32	−3120	−7250	−7·24	−3·09	+52500	−22400
2	2·32	−1630	−3780	−5·48	+4·59	+20700	−17360
3	2·32	−600	−1390	−3·41	+5·63	+4740	−7830
4	2·32	−70	−160	−1·16	+6·16	+190	−990

$P = -89380 \qquad M_x = +692530 \qquad M_y = +298820$

$$m_i = -\frac{89380}{42{\cdot}56} + \frac{692530}{1979}x + \frac{298820}{1095}y$$

$$= -2100 + 349x + 273y$$

$H_A = 127 \qquad H_D = 273$

$V_A = 451 \qquad V_D = 349$

Point	m_s	m_i	m	Point	m_s	m_i	m
A	−8800	−7560	−1240	5	0	−20	+20
B	−4000	−4250	+250	6	0	+630	−630
1	−3120	−3790	+670	7	0	+1060	−1060
2	−1630	−2760	+1130	8	0	+1270	−1270
3	−600	−1750	+1150	C	0	+1330	−1330
4	−70	−820	+750	D	0	−1980	+1980
E	0	−400	+400				

Therefore $M_B = 10{,}000(1 - \cos 53{\cdot}2) = 4000$ ft.-lb. The vertical component of the total load is $100 \times 8 = 800$ lb. and the horizontal component $100 \times 4 = 400$ lb. Therefore $M_A = 4000 + 400 \times 12 = 8800$ ft.-lb. The indeterminate moment m_i, and hence the final moment m, is calculated in *Table* V and the final bending-moment diagram is shown in *Fig.* 5.

Prestressed Piles 132 ft. long.

PRESTRESSED concrete piles 131 ft. 8 in. long, made by the pre-tensioning method, were used in the reconstruction of a pier in San Francisco, U.S.A. Tenders showed that these were cheaper than precast reinforced concrete piles or timber piles with gunited tops.

The piles are octagonal in cross section and measure 1 ft. 8 in. across the flats. They have a hollow core of 11 in. diameter except for solid lengths of 4 ft. 6 in. at the top and bottom. The piles are prestressed by eighteen high-tensile steel wires of ⅜ in. diameter. A prestressing bed 500 ft. long for making three piles at a time on each of four production lines was constructed fifty miles from the site, to which the piles were transported by barge.

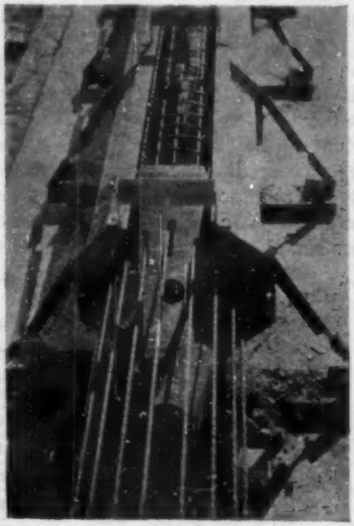


Fig. 1.

The prestressing wires passed through slots in the steel points of the piles (*Fig.* 1) and were tensioned simultaneously by an hydraulic jack of 200 tons capacity to a stress of 175,000 lb. per square inch.

It was specified that the concrete (tested on cylinders) should have a strength of not less than 5000 lb. per square inch at 28 days and not less than 4000 lb. per square inch before the force in the wires was transferred to the concrete. A modified rapid-hardening cement was used, and the piles were steam cured for 24 hours and driven three or four days after they were cast.

Each pile weighed 16½ tons and was lifted from four points of suspension 9 ft. 6 in. and 45 ft. 6 in. from both ends. They were driven by a floating rig with leads 120 ft. long, and the weight and fall of the hammer were such as to produce 32,500 ft.-lb. of energy. The 131 piles were driven to predetermined levels; this was possible because the level of the stratum of sand and gravel that supported the piles was known. The piles had a safe bearing capacity of from 60 to 70 tons, compared with the design load of about 50 tons. The piles were made and driven by Ben C. Gerwick, Inc., for the California State Harbour Commission. [The foregoing is abstracted from "Engineering News Record" for July 19, 1956.]

Facing a Dam by the Vacuum Process.

THE following describes the use of the vacuum concrete process for casting the concrete on the upstream face of Loch Quoich dam (*Fig.* 1), recently completed for the North of Scotland Hydro-Electric Board as part of the Garry hydro-electric works. The dam is 1100 ft. long and has a maximum height of 110 ft. A road 10 ft. wide extends across the top. The upstream face has a slope of about 1 in 1·3 and the downstream side 1 in 1·4. Watertightness is achieved by a concrete cut-off trench cut into the rock along the upstream toe of the dam and a reinforced concrete face 12 in. to 15 in. thick on the upstream face. The surface area of this face is 14,800 sq. yd. and, in order to reduce cracks due to shrinkage and settlement, construction joints are formed at a maximum spacing of 20 ft. in both directions; the joints have copper seals and a bituminous filler.

The face concrete was placed in two stages, the lower slabs comprising stage 1 and the remainder stage 2; this enabled impounding of the water to start before the top of the dam was finished.

For the first stage (*Fig.* 2) a 3-tons mobile derrick crane with an 80-ft. jib, travelling on a timber gantry halfway up the face was used. The concrete was delivered by lorries carrying skips of 1 cu. yd. capacity to the foot of the dam.

On the second stage two fixed 7-tons derrick cranes with 120-ft. jibs were used (*Fig.* 3). These were first erected on the top of the dam at one end and 90 ft. apart; as the work progressed one crane was used to dismantle and re-erect the other 90 ft. farther along the face. Only ten hours were required to dismantle and re-erect a crane and only one day's work was lost when a crane was moved. Concrete was delivered by locos. pulling bogies carrying 1-cu. yd. skips and travelling on a track just downstream from the top of the dam and extending its full length; at one end the track passed under a concrete mixing plant with a 1-cu. yd. mixer and two 1-cu. yd. hoppers with air-operated discharge doors, arranged so that two skips could be filled simultaneously.

In placing the concrete on the upstream face, timber side and end forms 12 in. deep, incorporating the copper seals between the slabs, were laid (*Fig.* 2) and the vacuum was applied to the top shutters. These shutters were 20 ft. long

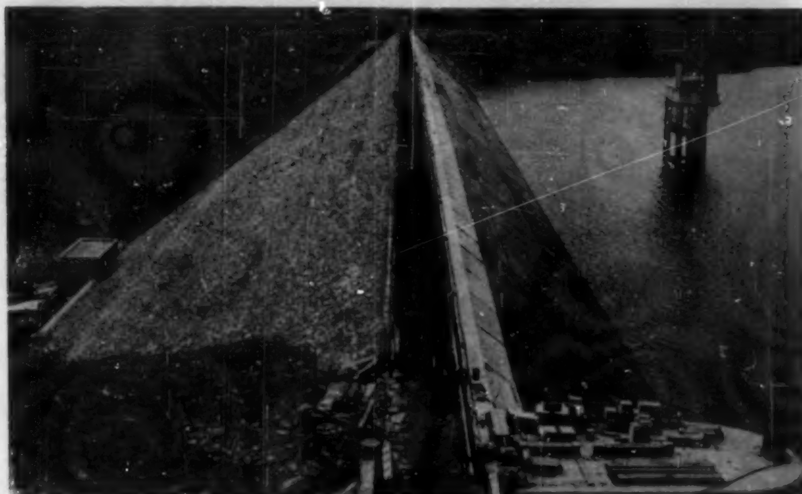


Fig. 1.—Loch Quoich Dam, Scotland.

by 2 ft. wide, and consisted of a waterproof plywood face ⅝ in. thick screwed to a frame of rolled-steel channel. Holes 1½ in. diameter were bored through the plywood at 3-ft. centres along the length of the shutter, and a steel pipe, 1½ in. diameter with flanged tee-pieces, inserted so that the tees coincided with the holes, was connected to the back of the shutter. One end of this pipe was blanked off and a vacuum-gauge screwed into the plug; the other end was fitted with a quick-release coupling for attachment to the vacuum manifold. The space between the frame and the plywood was filled with concrete to increase the weight of the

Fig. 2.—Concreting Lower Part of Face.

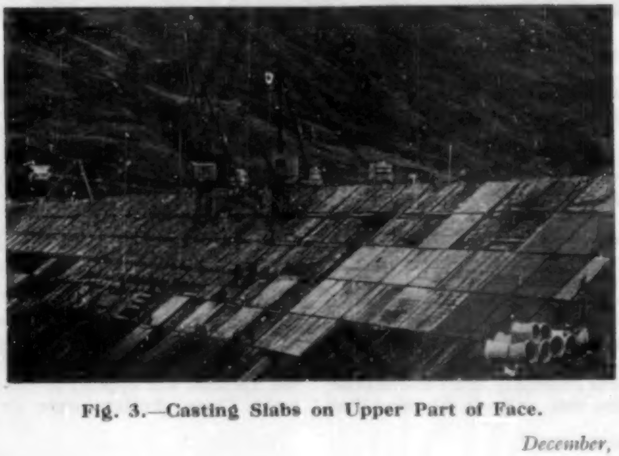


Fig. 3.—Casting Slabs on Upper Part of Face.

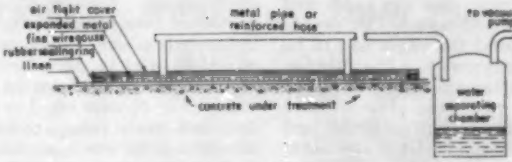


Fig. 4.—Arrangement of Vacuum Plant.

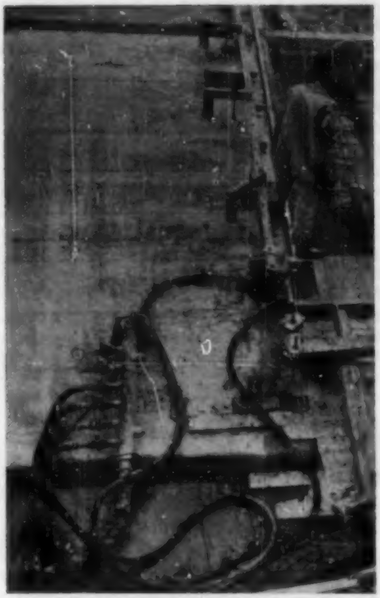


Fig. 5.—A Vacuum Shutter.

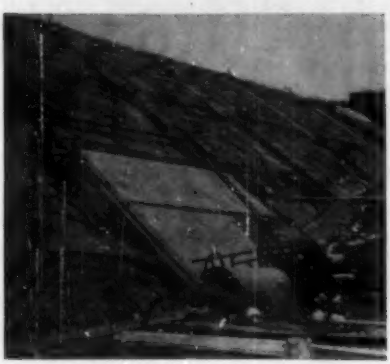


Fig. 6.—Vacuum Plant in Use.

shutter and to seal the joints in the pipe, only the two ends of the pipe being left exposed. The principle of the construction of a vacuum shutter is shown in *Fig.* 4. Two lifting eyes were fitted so that when the shutter was lifted it hung at the same slope as the face of the dam.

The front face of the plywood was sealed round its edges by timber strips 2 in. wide by ½ in. thick bedded in a bituminous material and nailed to the plywood. The small air-space required was formed by tacking a sheet of 16-gauge ¼-in. steel mesh to the face and this was covered with gauze. Finally, unbleached linen was stretched across the face and tacked into position.

Four shutters were used on each slab. The procedure was to place a shutter at the bottom of the slab to be concreted, fill it with concrete, and apply the vacuum; the next shutter was then placed above the first and the operation repeated until all four shutters had been filled. By this time the first shutter could be lifted and placed above shutter No. 4 and concreting continued, and a slab could be concreted in 2½ hours.

Vibration was used during the placing of the concrete and also for a short time after the vacuum had been applied; it was found that in addition to quickening the compaction of the concrete, the vibration helped to seal any small air leaks in the shutter. A vacuum reading of between 15 and 20 Hg at the shutter resulted in a satisfactory surface finish. Immediately after stripping the shutter the surface of the concrete was dry and dense, and generally it had no further treatment.

At the end of each day's work the shutters and the pipes were hosed. The linen covers were left in position unless they were to be renewed, in which case the cover was stripped off and the opportunity taken to clean the gauze and mesh. The average number of times a shutter was used before the linen had to be

renewed was twenty, and the mesh and gauze after about 180 uses. The timber sealing strips around the edges had to be replaced fairly frequently. The number of times each shutter was used during the work was about 750; the plywood face was still in good condition, and was used as shuttering on another part of the work.

During the second stage, eight shutters were made to enable two slabs to be concreted together. Four men were employed to each crane and were responsible for handling the shutters, placing the concrete, applying and timing the vacuum, and cleaning and maintaining the shutters; none of them had previous experience with this type of concrete, but after a short time became proficient.

The vacuum was applied by two rotary pumps with capacities of 348 cu. ft. per minute and 275 cu. ft. per minute and driven by 25-h.p. and 15-h.p. electric motors. During stage 1 the pumps were placed where they would need the least movement. During stage 2 two pumps were used near each end about halfway up the face and 700 ft. apart. They were linked together by a 3-in. screw-jointed main pipe with tees and valves about 80 ft. apart. Flexible pipes of 3 in. diameter led from these tees to the slab being concreted where a six-valve reduction manifold of 1½ in. diameter (*Fig.* 5) linked the shutters to the vacuum supply. Two tanks (*Fig.* 6) were fitted in the main pipeline close to the pumps to collect the water drawn out of the concrete. The pumps and pipeline gave very little trouble under normal conditions, but there was a great tendency for the water in the pipelines to freeze in cold weather; steam-heating the aggregate was helpful, and it was found to be important not to open the line to the atmosphere when the vacuum had been applied.

The rate of concreting varied considerably from month to month, due mainly to the weather; the best daily production was five 20-ft. by 20-ft. slabs by two teams of men in ten working hours, and the best monthly production by two teams was 99 slabs.

The consulting engineers were Sir William Halcrow & Partners, and the contractors Messrs. Richard Costain, Ltd. Millars Machinery Co., Ltd., are the licensees for the vacuum process in this country

The Cost of Multiple-story Buildings.

MR. PETER DUNICAN writes as follows.

It is surprising that in your November, 1956, number Mr. W. E. J. Budgen should suggest that in your October number Mr. Ove Arup, C.B.E., was arguing the universal validity of cross-wall structures for high flats. Of course he was not. Neither did the report of the Building Research Station make any assumptions in the matter—they stated only the conclusions they reached after studying a number of widely differing schemes. Cross-wall structures are not necessarily economical in themselves, but generally it has been found by even the most perverse and undisciplined planners that cross-wall structures lead to economical flats, and it is this result that matters.

Mr. Budgen makes a number of detailed points, all of which were thoroughly investigated by the design team set up by the London County Council to carry out the Picton Street development. The team consisted of architect, engineer, quantity surveyor, and contractor, and the definite conclusion they reached was that for this particular development, which consisted of four eleven-story blocks and sixteen four-story blocks, cross-wall structures in concrete produced the cheapest dwellings. It may well be that in some other schemes, with different planning requirements, a beam-and-column solution or a beam-column-and-wall structure may produce the best results. This is a matter for investigation in each particular case and is, in passing, a justification for considering the problem without prejudice. It would be a severe criticism of any planner if he allowed the structure to render his planning unsuitable for its purpose. In fact it would be a negation of planning.

Lightweight Aggregate from Pulverized-Fuel Ash.*

By W. KINNIBURGH, F.R.I.C.

PULVERIZED-FUEL ash is the residue from the combustion of powdered coal in modern boiler-furnaces used in many of the power-generating stations of this country. It is a grey powder resembling Portland cement in fineness and general appearance. Ash of this fineness is gas-borne, and is recovered from the flue-gas either by electrostatic precipitators or cyclone separators, from which it is obtained as a dry powder. The material consists of minute spherical glassy particles as shown in *Fig.* 1. These may be made to cohere by heat treatment producing porous pellets having considerable strength; an enlarged cross section of a pellet is shown in *Fig.* 2.

Fig. 1.—Photomicrograph of Ash.

Many tons of material suitable for use as aggregate in concrete have been produced at the Building Research Station by the sintering of pulverized-fuel ash, and tests have been carried out on this aggregate and on the concrete produced from it. Most of the work done so far has been on concrete suitable for block-making, as it was known that in the first place, at any rate, the aggregate would be required for this purpose. *Table* 1 gives a few examples of the properties of "clinker-type" blocks made with this aggregate compared with blocks made with a typical furnace-clinker aggregate. It will be seen that blocks made with pulverized-fuel ash aggregate are well within the requirements of the British Standard and have properties superior to the clinker blocks used in the tests. A small part of the aggregate was crushed in order to obtain the required grading.

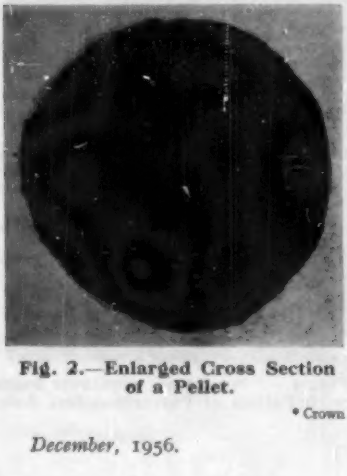


Fig. 2.—Enlarged Cross Section of a Pellet.

Whilst most of the work has been done on concrete suitable for blocks, some attention has been paid to structural concrete made with this aggregate, and *Table* 2 shows typical results obtained with various mixtures of vibrated concrete. A compressive strength of about 2500 lb. per square inch at a dry density of about 95 lb. per cubic foot can be expected using this aggregate, whilst compressive strengths up to 3500 lb. per square inch or more can be attained by the inclusion of sand, but with some sacrifice in lightness.

Sintered pulverized-fuel ash, in common with other lightweight aggregates, can be used to advantage for purposes

TABLE 1. PROPERTIES OF CONCRETE BLOCKS MADE WITH PULVERIZED-FUEL ASH AND CLINKER.

Type of aggregate	Density of graded aggregate (lb. per cu. ft.)	Proportions cement : aggregate (by volume)	Compressive strength at 28 days (lb. per sq. in.)	Dry density of concrete (lb. per cu. ft.)	Drying shrinkage (per cent.)
P.-F. Ash No. 1 . .	59·7	1 : 6	1000	76	0·047
		1 : 9	630	71	0·036
P.-F. Ash No. 2 . .	62·4	1 : 6	1050	80	0·047
		1 : 9	630	71	0·035
P.-F. Ash No. 3 . .	60	1 : 9	720	72	0·027
Clinker (20% combustible content) . .	60	1 : 6	860	86	0·063
		1 : 9	400	78	0·055
B.S. No. 2028, Type B blocks	—	—	Not less than 400	Not greater than 100	Not greater than 0·06

such as insulating concrete toppings on concrete roofs. Retention of water by lightweight aggregates frequently causes trouble if some of the mixing water becomes sealed between the base and the impervious top layer. These pellets have been treated with a bitumen solution to reduce the water absorption and such aggregate has been studied in the laboratory and on a roof. It has been found that the mixing water can be reduced to one-third of that normally used. *Fig.* 3 shows a roof covering being laid.

No-fines concrete made with pulverized-fuel-ash pellets has a remarkably high degree of workability due to the spherical shape of the pellets (*Fig.* 4).

The production of pellets from extremely fine powder is done by sintering at high temperature to bring about the cohesion of the particles in a state of incipient fusion. The air for combustion is blown through a permeable bed of material.

The ash inevitably contains some unburnt fuel, and it would be of prime importance if this fuel were sufficient to raise the temperature of the ash to the

TABLE 2. PROPERTIES OF VIBRATED STRUCTURAL CONCRETE

Proportions (by volume)	Water-cement ratio	Compressive strength (lb. per sq. in. at 28 days)	Dry density (lb. per cu. ft.)
1 : 6	1·40	1500	91·8
1 : 5	1·27	1850	93·7
1 : 4	1·02	2750	96·7

Fig. 3.—Levelling Concrete made with Pellets.

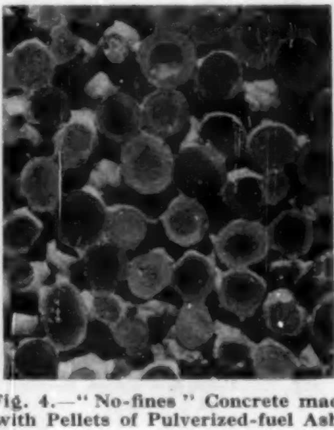


Fig. 4.—" No-fines " Concrete made with Pellets of Pulverized-fuel Ash.

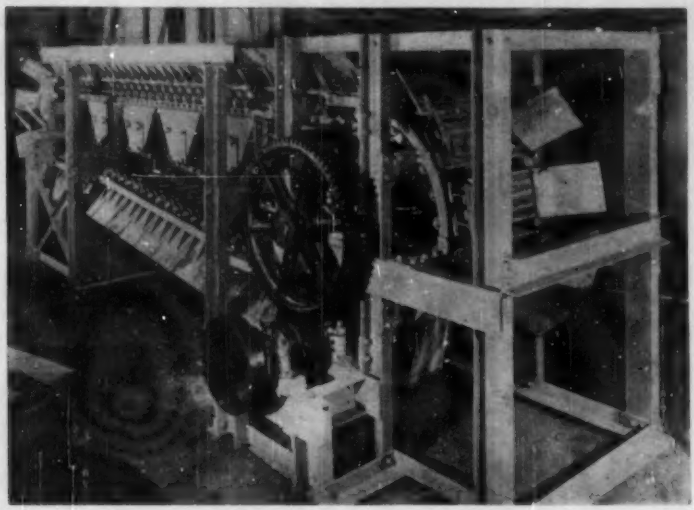


Fig. 5.—Pilot Sintering Strand.

Fig. 6.—Machining for Pelletizing Ash.

sintering point. Work done at the Building Research Station indicates that in a vertical shaft kiln (*Fig.* 7), in which there is good conservation of heat, ash with carbon content of about 3½ per cent. can be sintered without additional fuel; this is about the lowest of the range of fuel contents commonly met with in pulverized-fuel ash. On the other hand, it has been shown elsewhere that about 8 per cent. of fuel is necessary for sintering to take place on an open hearth, such as the sinter-strand shown in *Fig.* 5 (reproduced by courtesy of Huntington Heberlein & Co.); this value is in the upper region of fuel contents frequently encountered. It would seem that the choice between the shaft kiln and the open hearth depends on the prevailing fuel content of the ash but, however high the fuel content of an ash, the sinter-strand requires continuous ignition from an external source of fuel, which is not necessary with the vertical kiln. Moreover, the fuel content of the ash from a single source cannot be depended upon to remain consistently high enough for the ash to be used on a sinter strand for the period of years necessary for the amortization of this expensive plant, so that sooner or later it might become necessary to augment the natural fuel content of the ash by the addition of powdered coal.

In order to provide ash of sufficient permeability to permit the passage of combustion-air, it was considered necessary to convert the fine ash into pellets. This can readily be done in the rotating pan shown in *Fig.* 6. The inclined pan rotates slowly and the ash cascades down the sloping channel under a fine spray of water; uniformly spherical pellets are thus formed which are strong enough to withstand handling (a fairly wide range of size of pellet is possible on this machine). The pellets are fed to the shaft kiln, sinter-strand, or other furnace, where in a strong air current the integral fuel is employed to heat them to their sintering temperature (1000 deg. to 1200 deg. C.).

To summarize, it is possible to convert pulverized-fuel ash into a strong aggregate by pelleting without a binder (other than water) and sintering the "green" pellets by means of the residual fuel of the ash. The process is therefore fundamentally a cheap one: the labour cost should not be high but the capital cost of the plant is likely to be considerable. Several manufacturers are already doing development work and one firm is erecting a plant to sinter the entire output of pulverized-fuel ash from one of the principal London power stations, so that this aggregate may be available in the near future. The contributions made to this development are part of the research programme of the Building Research Station, and this paper is published with the permission of the Director of Building Research.

Fig. 7.—Vertical Kiln for Sintering Ash.

Design Based on Loads Causing Failure.

EXAMPLES OF DESIGN.

The following is an abstract of a paper entitled "Load-factor Design in Building Regulations: Future British Practice", presented by Dr. D. D. Matthews, M.A., M.Sc., D.Eng., to the symposium on "The Strength of Structures" held in London recently under the auspices of the Cement and Concrete Association.

Load-factor design is essentially the design of sections to carry "ultimate" loads, that is loads causing failure, equal to the design loads multiplied by a load factor. Load-factor design does not affect the determination of the moments and forces in continuous structures, which it is assumed will generally be calculated by the theory of elasticity. The method applies to the design of sections, not to external moments and forces.

In the elastic theory sections are so proportioned that when subjected to the design loads the stresses in the concrete and steel are specified safe fractions of their ultimate strengths. The difference between the two methods is due to the fact that the ratio of the design load to the ultimate load is not equal to the ratio of the design stress to the ultimate stress as computed by the elastic theory on the basis of a constant modular ratio. If these ratios were equal there would be no difference between the methods.

For under-reinforced sections, in which the ultimate strength is primarily determined by the reinforcement to resist tensile forces, the difference between the two methods is slight. For over-reinforced sections (which require reinforcement to resist compression as well as tension) there is a significant difference between the methods. This is due to the underestimation of the compressive capacity of the concrete by the elastic method, which leads to the unnecessary use or over-provision of reinforcement to resist compression in sections subjected to bending only as well as those subjected to combined bending and compression. The practical advantage of load-factor design in normal building construction is the elimination or reduction of reinforcement to resist compression. The load-factor method applies only to the tensile stresses in reinforcement and the compressive stresses in reinforcement and concrete. The extension of the method to shearing and bond stresses is not yet practicable.

The definition of load-factor design does not include the determination of the actions in continuous structures due to inelastic behaviour near ultimate loads. The investigation of the inelastic behaviour of reinforced concrete indeterminate structures has not reached the stage for inclusion in the Code of Practice. The terms "limit design" and "yield-line theory" have been used to distinguish this aspect of inelasticity from the concept of load-factor design.

The recent report of the Joint Committee of the American Society of Civil Engineers and the American Concrete Institute on load-factor methods uses the term "ultimate-strength design", which is also adopted in the current proposals for the revision of the Building Code of the American Concrete Institute. The main difference between these ultimate-strength design proposals and those proposed for the revised British Code of Practice No. 114 is that the British method proposes the adoption of different values of the load factor to be provided against failure of the steel and against crushing of the concrete instead of a single value for the two forms of failure.

If a section is designed for a load factor of 2 against a failure primarily due to yielding of the reinforcement and a load factor of 3 against failure due primarily to crushing of the concrete, then failure must occur at the lower load factor due to yielding of the reinforcement. The reason for specifying a higher load factor for crushing of the concrete is the extreme undesirability of this form of failure, coupled with a recognition of the great variability of the crushing strength of concrete. All sections designed according to the British proposals will fail primarily due to yielding of the steel provided that the concrete at the time of the test has two-thirds of its specified crushing strength. As this crushing strength is normally required at 28 days, the chance of failure due to crushing during the life of the structure is as remote as it is undesirable. The American proposals are restricted to members made with "controlled" concrete, whereas the proposals

for the revised British Code permit the use of load-factor design for all grades of concrete. This distinction justifies, at least in part, the less severe requirement against failure due to crushing of the concrete.

Rectangular Beams.

Rectangular sections commonly occur at the junctions of T-beams with columns, when the upper flange is in tension, and in beams crossing roof lights. The shape is ill suited to reinforced concrete, and the use of load-factor design is an advantage for this section, which serves as a convenient example for comparing the application of the two methods of design.

Consider a rectangular section designed for a modular ratio of 15 with stresses in the steel and concrete of 18,000 lb. and 1000 lb. per square inch respectively. The moment of resistance of both the steel and concrete is $193bd^2$ and the amount of reinforcement is 1·26 per cent. The calculation of the moment of resistance by the load-factor method starts by equating the force in the steel at its yield-point to the compression in the assumed rectangular "stress block".* The stress in the rectangular block when the steel is stressed to its yield-point is the cube strength multiplied by a factor (which has been determined experimentally to be $\frac{2}{3}$) relating the cube strength to the maximum attainable uniform compression in bending and multiplied again by the ratio of the load factors.

Thus the uniform compressive stress in the assumed rectangular "stress block" when the steel yields is

$$u \times \frac{2}{3} \times \frac{2}{3}.$$

For a permissible stress of 1000 lb. per square inch, u = 3000 lb. per square inch, so

$$\frac{4u}{9} = 1333 \text{ lb. per square inch.}$$

The depth of the stress block is thus calculated, as a fraction of the effective depth, for a yield stress of 36,000 lb. per square inch, as

$$1{\cdot}26 \text{ per cent.} \times \frac{18{,}000}{1333} = 0{\cdot}341.$$

The moment of resistance of the concrete is then

$$0{\cdot}341 \times 1333 \times (1 - 0{\cdot}17) = 378bd^2$$

*See page 581.

for a load factor of 1 against yielding of the steel and $\frac{2}{3}$ against crushing of the concrete, or

$$MR_c = 189bd^2$$

for a load factor of 2 against yielding of the steel and 3 against crushing of the concrete, which is slightly less than the value of $193bd^2$ given by the modular-ratio method. The actual load factor against yielding of the steel at the working load permitted by the modular-ratio method is

$$\frac{378}{193} = 1{\cdot}96,$$

which is slightly, if insignificantly, less than the ratio of the yield-point of the steel to the permissible stress of

$$\frac{36{,}000}{18{,}000} = 2.$$

The reinforcement required by the load-factor method for $193bd^2$ is 1·29 per cent. instead of 1·26 per cent.

The maximum moment of resistance of the concrete is determined in the load-factor method by assuming that the "stress block" extends half-way down the effective depth. For a load factor of 2 against yielding of the steel, the assumed uniform compressive stress is

$$\frac{1}{2} \times \frac{4}{9} \times u = 667 \text{ lb. per square inch,}$$

the lever arm is $0{\cdot}75d$, and the area of the "stress block" is $0{\cdot}5bd$, so that the maximum value of

$$MR_c = 0{\cdot}75 \times 0{\cdot}50 \times 667 = 250bd^2.$$

This increase in the capacity of the section requires more reinforcement to balance the increased compression. The reinforcement required is

$$\frac{250}{18{,}000 \times 0{\cdot}75} = 1{\cdot}85 \text{ per cent.}$$

instead of 1·26 per cent.

In order to resist a moment of $250bd^2$, compressive steel is required under the modular-ratio procedure, the amount depending on the depth at which the reinforcement is placed. If this depth is $0{\cdot}1d$, the required amount of compressive steel is 0·58 per cent. The tensile steel required is 1·61 per cent., which is less than in the case of the load-factor design, namely 1·85 per cent. So the total steel required is 0·58 + 1·61 = 2·19 per cent. for the modular-ratio method

and 1·85 per cent. for the load-factor method for identical concrete sections and moments. The extra reinforcement required by the modular-ratio method is 18 per cent.

When compressive steel is required with the load-factor method, the extra reinforcement in the modular-ratio method is even greater than in the previous case. For a bending moment of $350bd^2$ the compressive and tensile steel required in the modular-ratio method are 1·6 per cent. and 2·2 per cent. respectively, a total of 3·8 per cent. The corresponding amounts required in the load-factor design are 0·6 per cent. and 2·5 per cent., so that in this case the extra steel required by the modular-ratio method is 23 per cent. The modular-ratio design has an ultimate-load to design-load ratio of 1·76, even though the ratio of the yield stress in the steel to the design stress is still 2. The modular-ratio method in this case produces a 12 per cent. weaker section, even though it contains 23 per cent. more reinforcement, all of which is wasted in providing an unrealisable strength in the compressive zone.

The preceding calculations (for values of Q of 193, 250, and 350 with the qualities of steel and concrete in most common use) illustrate the general tendency of the modular-ratio method to provide too little steel in tension (because the lever arm at working loads tends to be greater than the lever arm at ultimate loads) and far too much steel in compression because of the restriction of the working stress to the stress in the surrounding concrete multiplied by a modular ratio. The resulting designs may be significantly if not seriously weak in tension and wastefully strong in compression.

T-sections.

Two types of T-section are commonly met with in practice, namely (1) T-beams cast integrally with floor and roof slabs and (2) sections used in ribbed and hollow-tile floor and roof slabs. The dimensions of the flanges of T-beams are determined by the load and the span of the slab, which very often result in a larger flange than is needed to resist the compression in the beam. In these cases the moment of resistance of the concrete is not a controlling factor, and the section is determined by the tensile moment of resistance of the reinforcement at the bottom of the section. In elastic design it is normal in these circumstances to neglect the compression in the concrete in the rib and to assume that the centre of compression lies at the mid-depth of the slab. This practice gives an identical lever arm, and therefore an identical amount of reinforcement, with those required by the load-factor method.

In ribbed and hollow-tile slabs the depth of the flange is not generally determined by other considerations. In these floors a minimum depth of section and flange is frequently an important requirement. In load-factor design the average compression in the flange is $0{\cdot}667c_b$ and is independent of the depth of the flange, whereas in elastic design the average compression in the flange, when $\frac{t}{c_b m} = 1{\cdot}2$, varies from $0{\cdot}5c_b$ for a flange half the effective depth to $0{\cdot}816c_b$ for a flange one-sixth of the effective depth. In the second case, when $\frac{d}{d_s} = 6$, the comparative moments of resistance of the concrete section are $0{\cdot}126c_b bd^2$ for elastic design and $0{\cdot}102c_b bd^2$ for load-factor design. This comparison neglects the compression in the rib below the slab and above the neutral axis in the elastic design, and above half the depth of the rib in load-factor design.

For $\frac{b}{b_r} = 6$, which is typical of ribbed construction, and for $\frac{d}{d_s} = 6$, the comparative moments of resistance of the concrete section are $0{\cdot}137c_b bd^2$ for elastic design and $0{\cdot}127c_b bd^2$ for load-factor design when the compression in the rib is considered. It is thus far more important to consider the compression in the rib for thin T-sections in load-factor design than in elastic design.

In the somewhat thicker sections that are typical of hollow-tile construction, the load-factor method gives higher moments of resistance for the concrete section. For example, with

$$\frac{b}{b_r} = 4 \text{ and } \frac{d}{d_s} = 4,\ MR_c = 0{\cdot}172c_b bd^2$$

for load-factor design against

$$0{\cdot}169c_b bd^2$$

for elastic design when the compression of the rib is considered. The comparative

figures when the compression of the rib is neglected are

$$0{\cdot}146c_b bd^2$$

for load-factor design and

$$0{\cdot}161c_b bd^2$$

for elastic design, which again illustrates the importance of considering the compression of the rib in the load-factor method; in elastic design the compression of the rib is $4\frac{1}{2}$ per cent. of the total moment of resistance, whereas in the load-factor method the compression of the rib contributes 15 per cent. of the total moment of resistance.

Columns.

In the discussion of rectangular beams it was observed that two-thirds of the cube strength is the maximum attainable uniform compression in bending. A similar ratio of two-thirds is also considered to be the maximum attainable average stress in an axially-loaded column. The direct compression in the British Standard Code No. 114 of 1948 for this type of column is $0{\cdot}76 \times \frac{1}{3}$ of the cube strength, so that the load factor against crushing of the concrete is

$$\tfrac{2}{3} \div (0{\cdot}76 \times \tfrac{1}{3}) = 2{\cdot}63,$$

which is the value proposed in the revised Code for axially and eccentrically-loaded columns.

The choice of a load factor of 2·63 against crushing of the concrete in columns, which is less than the value of 3 against crushing in beams, seems paradoxical if it is not appreciated that the lower load factor of 2 against yielding of the steel in beams always governs the ultimate strength. The load factor for the whole column when axially loaded depends on the amount of reinforcement; thus, for the minimum percentage of 0·8, a cube strength of concrete of 3000 lb. per square inch and a yield-point of steel of 36,000 lb. per square inch, the load factor is calculated as follows:

$$\frac{1 \times \frac{2}{3} \times 3000 + 0{\cdot}008 \times 36{,}000}{1 \times 760 + 0{\cdot}008 \times 18{,}000} = \frac{2288}{904} = 2{\cdot}53.$$

For the normal maximum amount of reinforcement, namely, 4 per cent., the load factor for axial loads is 2·32. So, in all cases of axial loading, the load factor of the section is greater than the load factor against yielding of the steel in beams.

The load factor for columns subjected to bending and compression depends on the eccentricity. For small eccentricities up to where failure is due to tension, the load factor decreases with increasing eccentricity from the value for an axially-loaded column (about 2·5) to the value against yielding of the steel (2). Above the eccentricity at which failure occurs by yielding of the tensile steel, the load factor remains constant at 2 up to the infinite eccentricity due to bending only.

In the treatment of beams, the load-factor method does not directly take account of the strains at ultimate loads. The restriction on the depth of the assumed rectangular "stress block" automatically prevents the assumption of incompatible strains with the steels in present use. It is not possible to avoid reference to the strains at ultimate loads in columns subjected to combined compression and bending. The new British Code proposals are that the assumed maximum concrete strain e_u at ultimate eccentric loading of a column should not exceed one-third per cent. This enables the depth of the neutral axis to be calculated for the load and bending moment producing simultaneous crushing of the concrete and yielding of the steel. For example, if $t_y = 36{,}000$ lb. per square inch and $E_s = 30 \times 10^6$ lb. per square inch, then the corresponding strain yield $e_y = 0{\cdot}12$ per cent. and the depth of the neutral axis is

$$\frac{e_u}{e_u + e_y}d = \frac{0{\cdot}333}{0{\cdot}453}d = 0{\cdot}735d.$$

In order to calculate the position and magnitude of the resultant compression in the concrete, the characteristics of the assumed "stress block" are needed. The Code proposals imply that

(1) The resultant compression is equivalent to a rectangular stress block extending to the neutral axis with an average stress of

$$\tfrac{2}{3}u \times \frac{\text{load factor against yielding of the steel}}{\text{load factor against crushing of the concrete}} \times 0{\cdot}85$$

$$= \tfrac{2}{3}u \times \frac{2}{2{\cdot}63} \times 0{\cdot}85 = 0{\cdot}431u.$$

(2) the depth of the resultant compression is 0·425 times the depth of the neutral axis.

The eccentric load which, when multiplied by the load factor (2) against yielding of the steel, produces yielding of the tensile and compressive reinforcement simultaneously with a concrete stress of $0{\cdot}431u$, may be termed the balancing eccentric load P_b.

Fig. 1 shows the relationships between the permissible load and the bending moment about the centre-line of a symmetrically-reinforced column 17 in. square for the two methods of design. The relationship for each method is drawn for a low percentage of reinforcement (0·83) and a high percentage of reinforcement (4·13). In all four cases the area of the compressive reinforcement has not been deducted in the calculation of the compression in the concrete. This is, of course, not a strict interpretation of the modular-ratio theory; it is hoped that

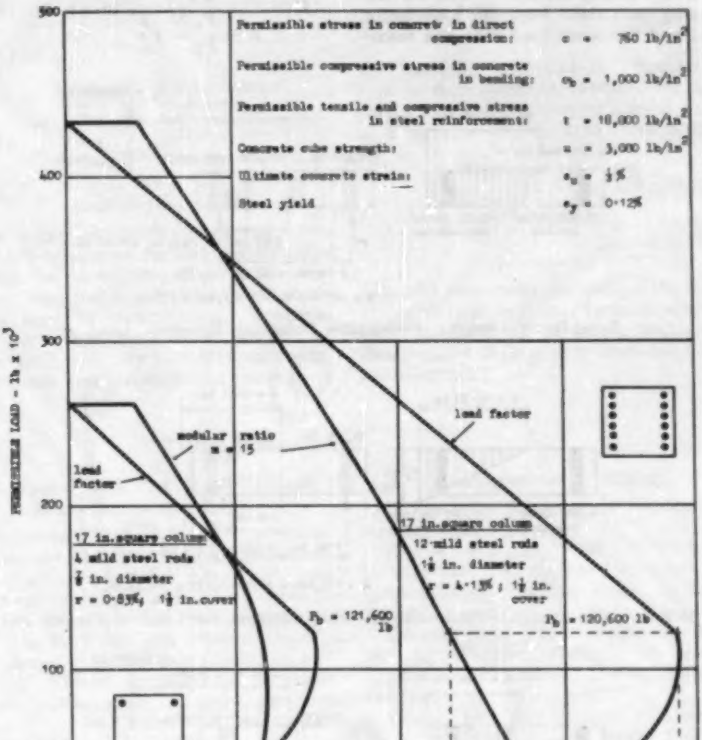


Fig. 1.—Relation Between Permissible Load and Bending Moment for Load-factor and Modular-ratio Methods of Design.

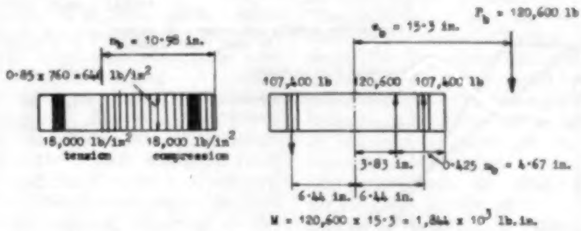
neglect of this over-meticulous refinement will be permitted in the load-factor clauses of the new Code.

The difference between the design curves is less for a small amount of reinforcement than for a large amount because it is due to the underestimation in the modular-ratio method of the ultimate compressive capacity of the reinforcement as well as of the concrete. The essential difference in the results of the two methods of design is shown in *Fig.* 2, which gives the magnitude and positions of the resultant forces acting on the section for the balancing eccentric load in load-factor design and the same load with the maximum moment permitted by modular-ratio design.

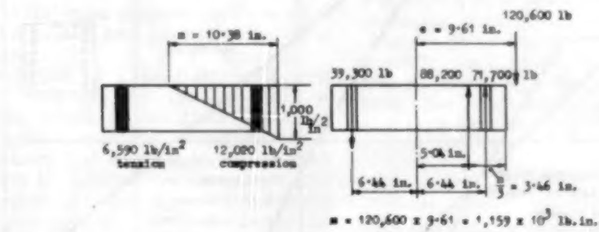
The load-factor curves consist of an assumed straight line between the axial load P_o and the balancing eccentric load P_b. Although this line is in good agreement with experimental results, it must be emphasised that it is not based on theoretical analysis ; such analysis leads to a cubic equation which is practically the chosen straight line.

The algebraic equation for the straight line is

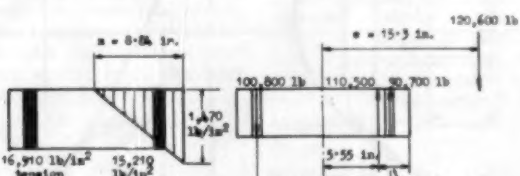
$$P = \frac{P_o}{1 + \frac{P_o}{P_b} - 1\frac{e}{e_b}}, \text{ in which}$$



Load-factor Design : Permissible Balancing Eccentric Load.



Modular-ratio Design : Permissible Bending Moment for Load of 120,600 lb.



Modular-ratio Design : Stresses for Load of 120,600 lb. e = 15·3 in.

REINFORCEMENT 4·13 PER CENT. COVER OF CONCRETE 1½ IN.

Fig. 2.—Column 17-in. Square.

$$P_0 = cBD + tA_s,$$
$$P_b = cBD \times 0{\cdot}85 n_b', \text{ and}$$
$$P_b e_b = cBD^2\left[0{\cdot}85 n_b'(0{\cdot}5 - 0{\cdot}425 n_b') + 0{\cdot}5 r d_1{}^1\left(\frac{t}{c}\right)\right],$$

where c is the permissible stress in concrete in direct compression,
t is the permissible stress in steel in tension and compression,
B is the breadth of the column,
D is the total depth of the column,

$$n_b' = \frac{\text{depth of neutral axis for } P_b}{D},$$

$$r = \frac{A_s}{BD} = \frac{\text{total area of symmetrical reinforcement}}{BD},$$

$$d_1{}^1 = \frac{\text{distance between reinforcement } (d_1)}{\text{total depth of column } (D)}.$$

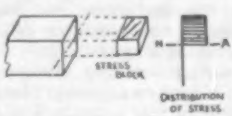
This equation applies only to symmetrically-reinforced rectangular columns in which the permissible stress in the steel in tension is equal to the permissible stress in the steel in compression. Practical cases for other conditions are better treated arithmetically from first principles.

For loads less than P_b the curves are derived from the conditions of equilibrium based on the characteristics of the " stress block " implied in the Code, and on the fact that the tensile and compressive steel both yield at ultimate loads for the whole range $P = P_b$ to $P = 0$. The resulting relation between P and e (the eccentricity about the centre-line of the column) is

$$P = cBD\left[(0{\cdot}5 - e') + \sqrt{(0{\cdot}5 - e')^2 + r d_1{}^1\left(\frac{t}{c}\right)}\right],$$

where $e' = \dfrac{e}{D}$.

[The term " stress block " denotes the volume that would be obtained by multiplying the stress acting on a cross section of a member by the area over which it



acts and represents the compressive force acting at that section. In most ultimate-load methods of design a rectangular distribution of stress is assumed, as in the diagram.]

The Associated Portland Cement Manufacturers, Ltd.

SIR GEORGE EARLE, C.B.E., is relinquishing the Chairmanship of the Associated Portland Cement Co., Ltd., on 31 December, 1956, from which date he will be President of the Company. Mr. J. A. E. Reiss (the present Deputy Chairman) will be the new Chairman and Mr. V. C. Ellison (at present a Director) will be a Managing Director and also Chairman of the Cement Marketing Co., Ltd., in place of Mr. Reiss.

Lectures on Building.

ARRANGED by Ministry of Works. Admission free.

The R.I.B.A. Form of Contract. By D. Gardam. North Staffordshire Technical College, Cauldon Place, Stoke-on-Trent. December 11. 7.15 p.m.

Flooring and Flooring Materials. By W. J. Warlow. College of Further Education, Newtown Road, Hereford. December 12. 7.15 p.m.

Mixing and Distribution of Concrete. By A. G. Stone. S.W. Essex Technical College, Forest Road, London, E.17. December 13. 7 p.m.

Construction of Dams in Scotland.

To provide 600,000 tons of aggregates for two dams—one at the outlet of Loch Cluanie and the other at the outlet of Loch Loyne—for the Glen Moriston hydro-electric scheme now nearing completion for the North of Scotland Hydro-electric Board, a granite quarry with a capacity of 200 tons an hour was opened at the site. A 25-in. primary gyratory crusher, a 4-ft. secondary gyratory crusher, and six double-rotor hammer-mills were installed. Rock blasted from the face was loaded by two 2-cu. yd. shovels into 4½-cu. yd. shuttle dumpers, which transported the rock a short distance to the crushing plant. The crushed rock passed through screens to storage bunkers, and thence by conveyor to the batching plant. The maximum size of aggregate was 3½ in. For the first time in this country Trief cement was used (as described in this journal for August 1954).

The storage at the batching plant (*Fig.* 1), which was near Cluanie dam where most of the concrete was required, consisted of a hexagonal bin of 200 cu. yd. capacity with six compartments for aggregates and a compartment to contain 50 tons of cement in the centre supplied from two silos each of 100 tons capacity. Tilting-drum pneumatically-controlled mixers, two with a capacity of 2 cu. yd. and one of 1 cu. yd., were used; the 1-cu. yd. mixer supplied either the block-making plant or the dams as required.

Instead of using normal shuttering for the faces of the dams, patent precast blocks with a face area of about 25 sq. ft. formed a permanent shutter, as briefly mentioned in this journal for January 1955. Each block weighed between 2 and 3 tons. About 500 blocks were cast in a shed each week and kept under cover for 24 hours, after which they were stored out of doors for 28 days before being used. Steel moulds were used, and the blocks were cast on a vibrating table designed by the contractors. The blocks were fixed roughly in position by mobile cranes, then jacked to the correct batter, and wedged in position with precast concrete wedges. The concrete for the dam was placed directly behind the blocks. It is seen in *Fig.* 2 that the blocks have dovetailed projections on the inner face to key with the backing concrete and with each other.

For the construction of Cluanie dam,

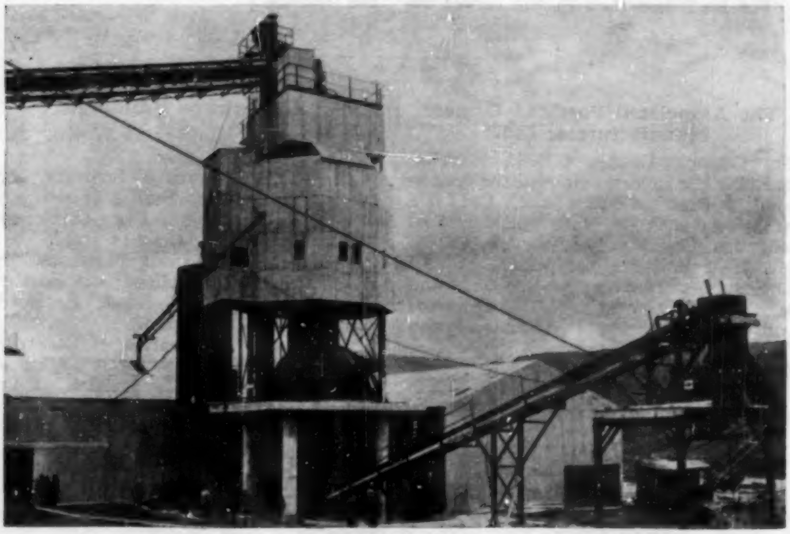


Fig. 1.—Mixing Plant, Aggregate Conveyor and Concrete Conveyor.

which is about 2300 ft. long, two electrically-operated luffing cableways (*Fig.* 3), each of 10 tons capacity, were used to transport the 210,000 cu. yd. of concrete, the facing blocks, and other materials. From the mixers concrete was delivered by conveyors to mobile hoppers which took it to the 4-cu. yd. skips which were carried by the cableways to the point of placing in the dam. The concrete plant had a capacity of 80 cu. yd. per hour. A return trip from the batching plant to the centre of the dam could be carried out in five minutes.

The cableways had a span of 2133 ft., the head and tail masts being 180 ft. and 160 ft. high respectively. The maximum depth of lift of the hook below the crest of the dam at the centre of the span was 300 ft., and the minimum height of lift of the hook above the crest was 44 ft. when fully luffed, the range of luff being 25 ft. on each side of the centre-line of the cableway. The "bounce" of the skip at the centre of the span when the concrete was discharged was about 15 ft. vertically. A 250-h.p. electric motor operating at 730 revolutions per minute gave a hoisting speed of 250 ft. per minute and a travelling speed of 1000 ft. per minute.

Loyne dam, which is about 1800 ft. long and contains 60,000 cu. yd. of concrete, was constructed by a different method. Instead of using cableways, electrically-driven derrick cranes, two of 10 tons capacity and one of 5 tons capacity, with 120-ft. jibs were used. The cranes placed the precast blocks accurately in position in addition to placing the concrete. This dam is about four miles from the mixing plant, and concrete was transported to the dam in truck mixers of 4½ cu. yd. capacity which deposited the concrete into 2-cu. yd. bottom-opening skips which were hoisted by the cranes. In this way about 25 cu. yd. per hour were placed.

For both dams the concrete mixtures used and the compressive strengths at 28 days were as follows. Precast blocks: 1 : 4; 3400 lb. per square inch. Upstream 4 ft. of dams: 1 : 4; 3400 lb. per square inch. Hearting: 1 : 7 (Trief cement); 1400 lb. per square inch.

The consulting engineers are Sir William Halcrow & Partners, and the Mitchell Construction Co. are the contractors.

Fig. 2.—Face-blocks in Position.

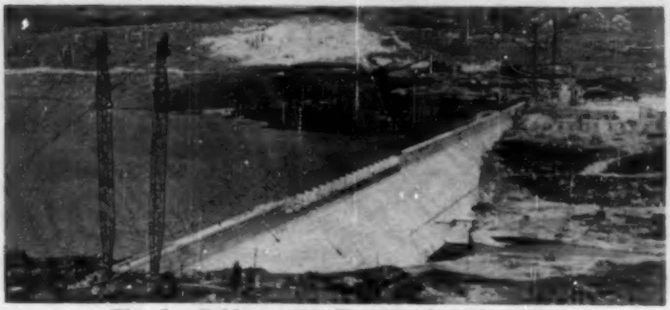


Fig. 3.—Cableway for Transporting Concrete.

Runways at Airfields.

THE following is abstracted from a paper presented on October 17 to a meeting of the Pavings Development Group of the Cement and Concrete Association by Mr. F. R. Martin, Superintending Civil Engineer of the Pavement Design, Experimental, and Development Branch, Directorate-General of Works, of the Air Ministry.

Riding Quality.

Concrete and good riding quality are rarely associated by the motorist or the aircraft pilot because many acres of roads and runways have been constructed without sufficient thought for the comfort of the user. While the need for relatively closely spaced joints in thin slabs is recognised, and random cracking through warping has been almost eliminated, little attention has been directed to riding quality at the joints. The aims have generally been quantity of output and test cubes with a high average compressive strength at twenty-eight days. Seldom has the aim been a smooth passage for the motorist or pilot. So far as aircraft with relatively low tyre pressures are concerned, it has not mattered a great deal. Quality of workmanship and pride in the finished article have been somewhat neglected, and it is not surprising that bituminous surfaces are generally preferred. To-day, however, emphasis is being placed on surface tolerances to provide smooth surfaces; tolerances of ⅛ in. variation in 10 ft. with *q* values of 30 in. per mile are the desirable limits for aircraft with tyre-pressures so high that they are virtually solid, and whose undercarriages need an even surface on which to land and take off.

The use of the scraping straight-edge has done more to produce good riding quality than any other item of equipment. The use of such a tool was introduced from the U.S.A.,* and with its use British contractors find that tolerances of ⅛ in. in 10 ft. are possible. The straight-edge (*Fig.* 1) is 10 ft. long, having sufficient weight and rigidity to act as a cutting and smoothing tool while maintaining its shape. The straight-edge is placed on the slab at the near edge and, with the handle at knee height, is pushed to the far edge; the handle is then lifted to shoulder height and pulled to the near edge. The straight-edge is moved half its length along the slab and the process repeated. The surface of the concrete can hardly fail to have the specified surface tolerance.

* This tool was illustrated and described in this journal so far back as August, 1949.—Ed.

Fig. 1.—Use of Straight-edge.

Consistency of Concrete.

When the straight-edge is used the concrete must be fairly workable. There has therefore been a breakaway from the use of the stiff, lean, harsh mixtures that produced high cube strengths, but often left honeycombing in the slab and resisted a satisfactory surface finish. Let us see no more of the 1 : 8 and 1 : 9 mixtures with no slump. Let us rather use mixtures of about 1 : 6 by weight, keep the water-cement ratio at about 0·5, and have mixtures sufficiently plastic to be brought correctly to level.

Joints.

Joints need no longer interfere with smooth riding, for after a joint has been formed in the plastic concrete it can be filled to the surface with a board and the scraping straight-edge used to level the surface across it. The top few inches of the board can later be cut out to a depth of a few inches for sealing. With this method there is no need to form bull-noses on the edges of any but expansion joints which require to be sealed below the surface, or to disturb the concrete after the straight-edge has been used. Perhaps, too, a ban could be placed on the use of small trowels.

Fig. 2.—Cutting out Filler-board.

When limestone aggregate is used, sawn joints are generally economical and now commonly used. With igneous rock aggregate, and more so with flint-gravel, it is less likely that cutting, even if it does not produce a jagged edge, will be economical. Whatever the aggregate, all saw cuts should be made with saws having one or more diamond or silicon-carbide circular blades; they should be operated by engines of at least 25 b.h.p., and cooled during cutting with an abundant supply of cold water. The joints should be cut before any random cracking of the concrete occurs. It is difficult

Fig. 3.—Sealing with Hot Compound.

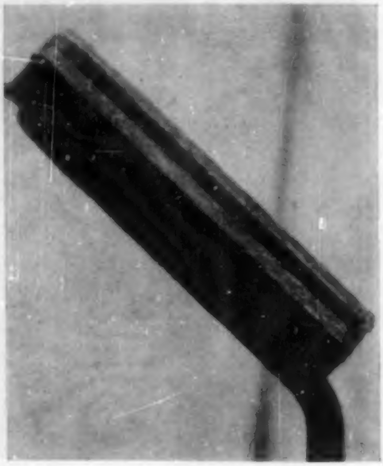


Fig. 4.—Hot Smoothing Tool.

Fig. 5.—Hot Smoothing Tool.

to judge the right time to cut with any but limestone aggregate, which is not so sensitive to temperature changes as is gravel; it may be any time between about eight hours and three days after laying. Random cracking probably always occurs to some extent, but it may be that some cracks can be accepted provided that they are not too numerous; this view appears to be held in the U.S.A., where joint sawing is adopted much more than in this country.

For the sealing of joints two compounds have been used extensively, namely bitumen-rubber for slabs not subjected to jet fuel and pitch-rubber for areas subjected to jet fuel. None of the materials so far developed is entirely resistant to attack by heat and blast from jet engines. The problem has been overcome to some extent by finishing the seal below the top of the slab. *Fig.* 2 shows the method commonly used for cutting out the filler board where it extends to the top of a joint. *Fig.* 3 shows subsequent sealing with hot compound. A final ironing of the seal with a heated iron bar (*Figs.* 4 and 5) appears to give the best results. It is important that the material should not overflow on to the surface.

A method of sealing joints with strips of plastic material has been investigated and trial areas have been laid to test its

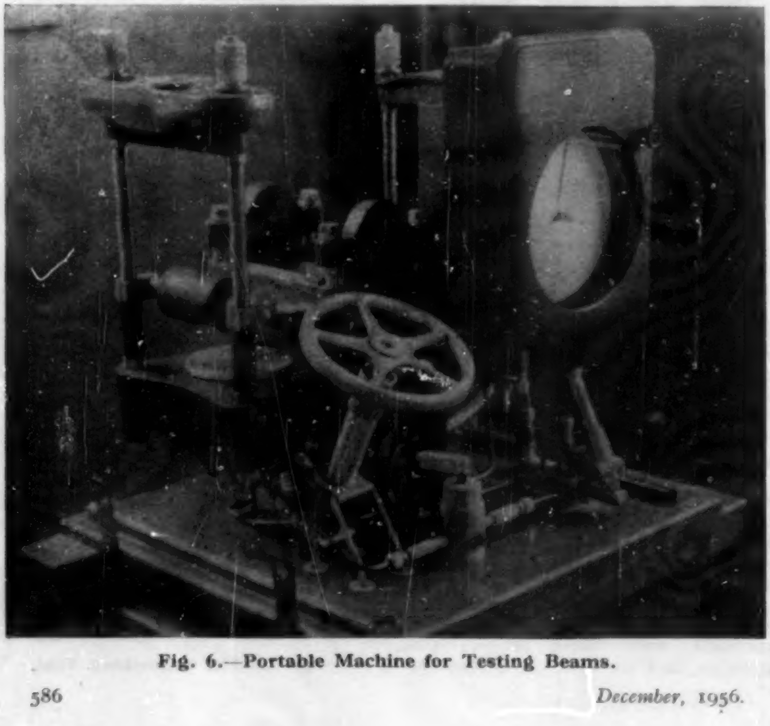


Fig. 6.—Portable Machine for Testing Beams.

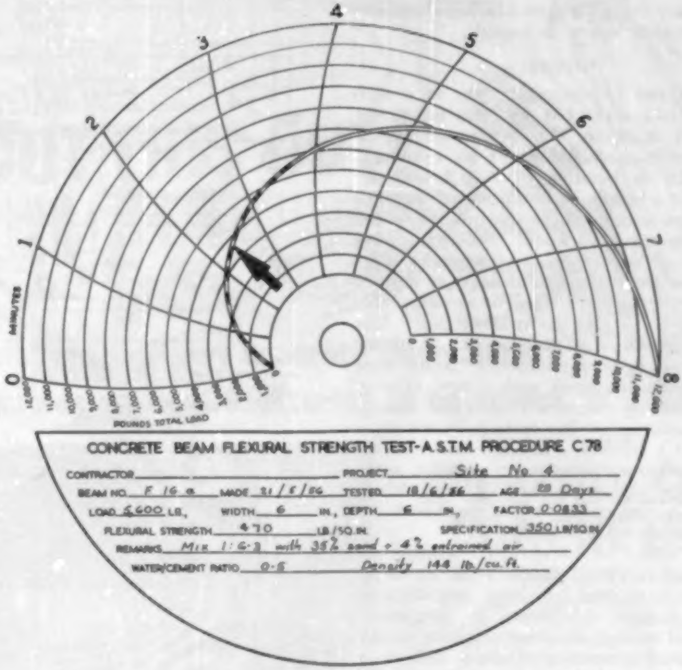


Fig. 7.—An "Automatic" Recording Chart.

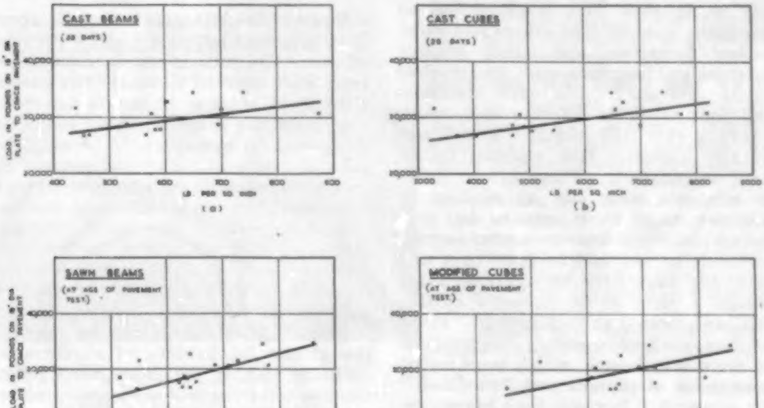


Fig. 8.—Effect of Bending Strength on Load-carrying Capacity.

efficiency in cold weather; a very uniform joint-sealing space is needed.

Strength.

Sufficient experience exists to design a concrete slab that will not fail under present or future known aircraft traffic.

Specifications require a minimum strength in bending of test beams instead of a minimum compressive strength of cubes as a guide to the quality of the concrete in the work. Six-inch cubes tested at seven and twenty-eight days have for long been the measure of quality and uniformity, yet designs are generally based on strength in bending. The logical measure of the strength of beams has now been introduced. Some sites have been equipped with a new type of beam-testing machine (*Fig.* 6) of American manufacture for testing beams on a span of 1 ft. 6 in. The machine is portable, and the strengths are recorded automatically on a chart (*Fig.* 7). What better record could an engineer wish for than a folder of these charts?

The effect of strength in bending on the load-carrying capacity of an 8-in. unreinforced slab has been investigated in a full-scale experiment by measuring the load needed to cause cracking of the corners of a number of slabs laid on a uniform base of Westergaard *k* (modulus of sub-grade reaction) of about 300 lb. per square inch per inch. The slabs were made with four combinations of aggregates, namely flint gravel and sand, crushed limestone and sand, crushed limestone and limestone sand, and crushed granite and sand. For each combination, four types of mixture were used, namely a rich stiff mixture, a rich wet mixture, a leaner stiff mixture and a leaner wet mixture. These sixteen different mixtures were laid at random to form two bays 20 ft. square for each mixture and eight free corners for testing. The concrete was laid in June 1955 and tested one year later by applying loads through a plate 18 in. diameter until a crack was formed at each corner. From each bay one broken corner was removed and sawn into beams, which were tested for modulus of rupture and "modified" cube strength. Beneath each corner removed, *k* tests were made to ascertain the uniformity of the base. *Figs.* 8 (a) to (d) give some of the results. It seems that although any of the samples gives

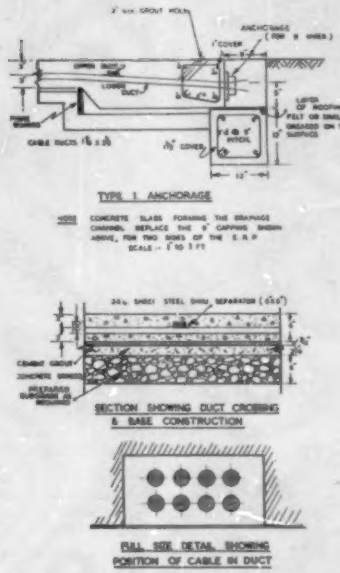


Fig. 9(a).—Prestressed Platform.

a good guide to the strength of the slab, the best guide is the bending strength of a beam sawn from the slab and tested at about the same age as the slab.

Air-entrained Concrete.

Some of the mixtures contained agents that entrained between 3 and 5 per cent. of air. The use of air-entrained concrete with internal vibration has enabled slabs 16 in. thick to be laid in one operation, with the advantage that joints can be spaced farther apart. The entrained air also aids workability and permits the production of concrete with good strength and durability. Air-entraining has also helped the change from harsh mixtures, and is another aid in the production of smooth and even runways. Mixtures require to be examined, and perhaps both the sand and water contents adjusted, so that full advantage can be taken of the greater workability of air-entrained concrete and at the same time reduce the loss of strength due to the entrained air.

Surface Finish.

Much work remains to be done on the relationship between the wear of aircraft

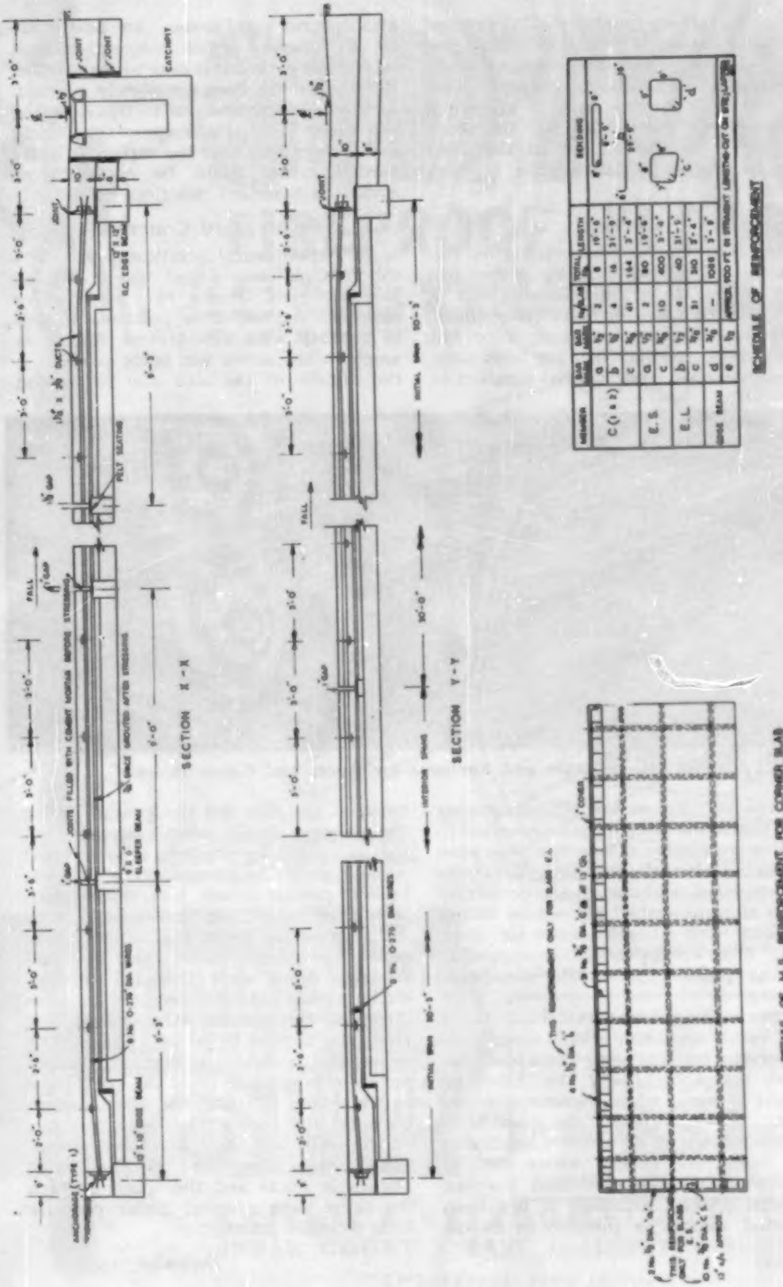


Fig. 9(b).—Details of Prestressed Concrete Platform.

tyres and surface roughness. At present the riding surface is produced by dragging a stiff broom across the concrete while it is plastic. The effect, compared with the rise of a canvas belt, is to give a better surface in wet weather on which an aircraft can brake, and at the same time to produce a much more uniform surface.

Curing.

The need for efficient curing is important. Membrane curing alone has been found to be satisfactory over a large part of the year. At other times, particularly on very cold or very hot windy days, the slab has not been completely covered, and this has resulted in hair cracks. To ensure efficient curing at all times it is thought to be essential to apply a membrane curing spray as soon as possible after the finishing operations are completed, and also to use protective covers of whitewashed hessian on frames and keep the surface damp for some days; these covers should be applied as soon as possible after the membrane spraying.

Some experiments have been made with membranes into which aluminium powder or white pigments have been dispersed, following closely the American practice of using white pigmented membranes. Whether it will be possible to dispense entirely in all weather conditions with protective covers where highly-reflective membranes are used has not yet been proved, although it has been suggested that it is possible in humid atmospheric conditions. An advantage of an aluminised or white-pigmented membrane is that it is easy to see whether the surface has been completely covered. A power-driven machine to travel on the side-forms so that the spray completely and uniformly covers the surface is badly needed; this could be copied from American standard practice.

Prestressed Concrete.

An experimental jointless prestressed concrete platform about 200 ft. square has been laid during the past twelve months. Attempts to prestress a slab in contact with the ground results in much of the stress not being present in the middle of the slab due to friction between the slab and the ground during the process. In the scheme shown [*Figs.* 9(*a*) and 9(*b*)] the concrete is prestressed while it is off the ground. The slab was built of precast panels, 6 in. thick, which were lifted by vacuum suction-pads on to walls projecting about ¾ in. above a prepared base (*Figs.* 10 to 13). The prestressing wires were threaded through ducts formed at 3 ft. centres in both directions through the slabs and the prestress was applied by jacks. The method of wedging is shown in *Fig.* 14. Special panels were made to resist the extra force at the edges. During the tensioning of the steel the panels were able to move on the walls to ensure an even distribution of stress throughout the whole area. The cable ducts and the space beneath the slabs were grouted under pressure with colloidal grout.

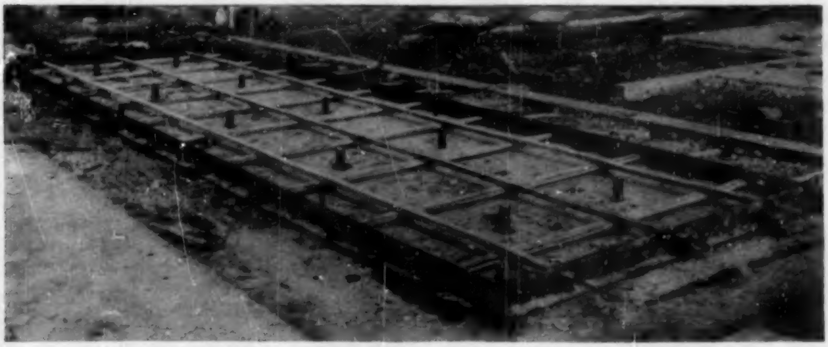


Fig. 10.—Shutter and Formers for Ducts and Grout Holes.

The slab has not yet been tested but, with a prestress of about 250 lb. per square inch and a bending strength of 600 to 800 lb. per square inch, it is expected that it will resist an unlimited number of applications of loads of at least 50,000 lb. on a plate of 12 in. diameter.

In the course of this work it was found that the vacuum process resulted in variations in thickness of slabs, due to some extent to variable amounts of water removed and also to the movement of the pad on the surface when the suction was released; to overcome this it will be necessary to dispense with the vacuum process or to cast the slabs vertically.

The rectangular ducts required rectangular wire spacers and caused difficulties in the threading of the wires; this will be overcome by using ducts of circular section. The threading of the wires in the ducts disturbed the metal "shim" plates fixed between the duct formers where they crossed each other, and grout escaped in a few places from

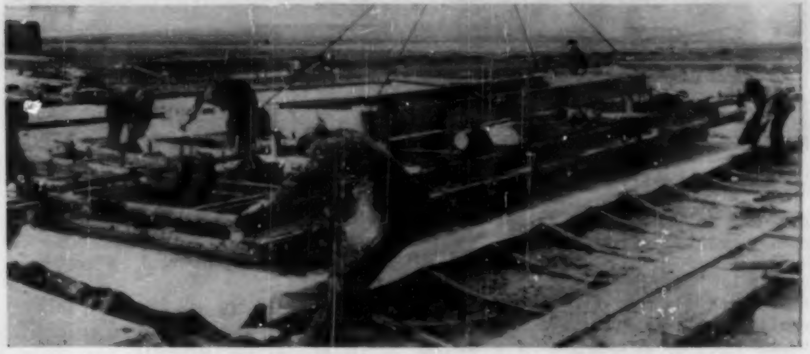


Fig. 11.—Lowering Suction-pad into Position.

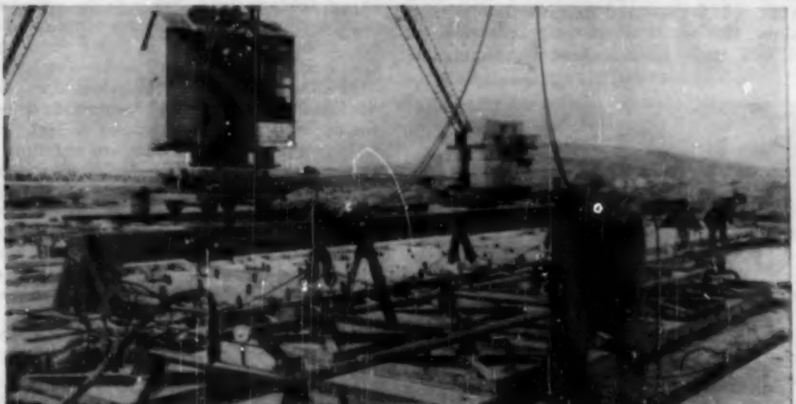


Fig. 12.—Suction-pads in Place ready for Lifting Slab.

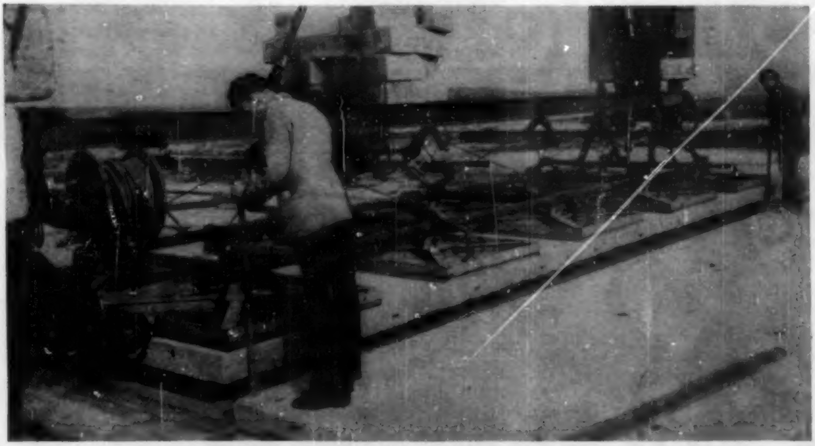


Fig. 13.—Lowering Slab on to Walls.

one duct to another; the use of circular ducts should avoid this trouble.

The main objects of the experiment were to see whether the idea of prestressing a slab while it was off the ground could be used in practice and, secondly, whether a slab so constructed would be as strong as was calculated and would maintain its strength at all times of the year. It is expected that, for heavy-duty slabs, the high cost of the process will be off-set to a large extent by saving in excavation, in joints, and in quantities of materials. It is unlikely, however, that it will be economical for roads.

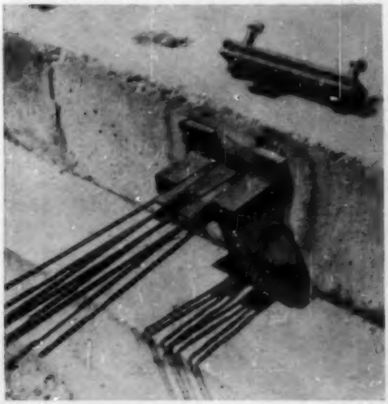


Fig. 14.—Method of Wedging Wires.

Testing.

Generally it has been accepted that an unreinforced slab fails when it cracks. Failure has been measured by applying an increasing load through a circular plate until a crack is formed; by applying a factor to the breaking load, a safe working load is assessed. For prestressed concrete, and probably also for reinforced concrete, this method does not apply, for simple cracking of slabs is no longer necessarily a measure of failure. The permanent movement of the surface runways under repeated application of the working load would seem to be the best measure of performance, and it may be that for unreinforced slabs also repeated loading within the assessed capacity, and the effects of fatigue, may be more damaging than has been thought. A programme of testing of unreinforced runways by repeated loading has therefore been arranged and will include prestressed and reinforced slabs. The strength will be assessed from the load that can be carried, say, 1000, 10,000 or 40,000 times, with a fixed maximum deflection or a permissible permanent settlement while still maintaining the riding quality of the surface.

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M-W.813

SITUATIONS VACANT. Assistant engineers (structural) required by consulting engineers in their Nigeria office for road and bridge works. Age about 20 to 30. 15 months' tour with three months' paid leave, free furnished accommodation, and passage to and from Nigeria. Good salaries paid, depending on experience. Permanent position after first tour either Nigeria or United Kingdom. Good prospects for self-reliant men. Also assistant engineer experienced in water supply for small townships, required for design and administration work in the same area and on the same conditions. Apply R. TRAVERS MORGAN & PARTNERS, 11 Victoria Street, London, S.W.1.

SITUATION VACANT. Assistant to resident engineer required at once for reinforced concrete viaduct in London area. Must have some previous site experience and should preferably have passed Parts I and II of the A.M.I.C.E. examination. Salary: £700 per annum. Apply HOWARD HUMPHREYS & SONS, Consulting Engineers, Victoria Station House, Victoria Street, London, S.W.1. Please quote reference 364.

SITUATION VACANT. Structural engineer required by London consulting engineers for concrete design department. Applicants must have first-class Honour's degree and at least 10 years' experience in the design of reinforced concrete and prestressed structures. Detailed knowledge of higher mathematics, of the design of indeterminate structures and the latest methods of design essential. Salary commensurate with responsibilities attached to the post. Apply in writing, with full particulars of age, qualifications and experience, to Box No. 551, c/o CHARLES BARKER & SONS, LTD., Gateway House, London, E.C.4.

SITUATIONS VACANT. Senior and junior detailer draughtsmen required by London consulting engineers. Applicants should be good draughtsmen with considerable experience in reinforced concrete work. 5-days' week. Superannuation scheme. Luncheon and sports clubs. Reply by letter, with full particulars of age and experience, to Box No. 550, c/o CHARLES BARKER & SONS, LTD., Gateway House, London, E.C.4.

SITUATION VACANT. Structural engineering representative required for the North of England. Applicants should be age 35 upwards and must be Corporate Members of the Institution of either Civil or Structural Engineers. Applicants must have extensive experience in travelling and relationship with important building clients and architects for large structural works. Salary will be not less than £2000 per annum, according to experience and status of applicant. Applications, which should give fullest possible details, will be treated in absolutely strict confidence, and should be addressed to Box No. 65, c/o HOLMES SON AND POTT, Solicitors, 3 London Wall Buildings, London, E.C.2.

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Vacancies for STRUCTURAL ENGINEERS in the DISTRICT SURVEYORS' SERVICE. Duties include checking of calculations and/or supervision of works in progress. Structural knowledge essential. GRADE II (salary on scale £987 to £1184), GRADE IIIS (up to £987), and ASSISTANTS (up to £818), with starting rates according to qualifications and experience.

Particulars and application forms obtainable from THE ARCHITECT (AR/EK/DS/5), County Hall, S.E.1. (2082.)

PORT OF LONDON AUTHORITY

The Port of London Authority have vacancies in their Drawing Office at Head Office, Trinity Square, E.C.3, for TECHNICAL ASSISTANTS of various grades, in the salary range of £645 to £956 per annum, for structural engineering designers and detailers.

Minimum qualifications are the holding of the Ordinary National Certificate in appropriate subjects and at least 3 years' experience in a design drawing office. Higher National Certificate is required for the higher grades. Experience in the design and detailing of reinforced concrete is essential, but knowledge of steelwork design will be an advantage.

It is the Authority's policy to encourage Technical Assistants to study for higher qualifications and facilities are provided accordingly.

Applications, giving age, qualifications and full details of experience, should be addressed to the ESTABLISHMENT OFFICER, Port of London Authority, Trinity Square, E.C.3.

SITUATIONS VACANT. Draughtsmen. Several vacancies exist for good intermediate and junior draughtsmen in precast reinforced concrete design office. Time off for technical school if required, and training given to juniors. Good salaries and prospects of advancement. Luncheon vouchers. Apply to MANAGING DIRECTOR, R. P. EAGAN, LTD., 167 Victoria Street, London, S.W.1.

SITUATION VACANT. Assistant engineer required by leading precast concrete manufacturers. Experience of reinforced concrete essential. Progressive position. Pension scheme. Five-days' week. Box 4400, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Reinforced concrete designer for work in Victoria Street, London. Five-days' week, non-contributory pensions scheme after three years' service. Progressive position. Five years or more experience with reinforced concrete specialists preferred. Write, giving full particulars, to THE SPENCER WIRE CO. (DEVELOPMENTS) LTD., 53 Victoria Street, London, S.W.1.

SITUATION VACANT. Reinforced concrete designer-draughtsman required. Familiar with frame structures and general design to Code of Practice. Good prospects, five-days' week, luncheon vouchers, and pension scheme. Apply, stating experience and salary required, to JOHNSON'S REINFORCED CONCRETE ENGINEERING CO. LTD., Artillery House, Artillery Row, London, S.W.1.

SITUATION VACANT. Junior structural engineering assistant, A.P.T.I. (£543 5s. to £625 5s. plus £65 Westminster Weighting, plus London Weighting £30 at 26 years and over, £20 at 21-25). To assist in detailing of frame structures in reinforced concrete and structural steelwork, and preparation of steel schedules. Commencing salary according to ability and experience. Established and pensionable, subject to prescribed conditions. Five-days' 38 hours' week. Application forms, stamped addressed foolscap envelope from COUNTY ARCHITECT, 1 Queen Anne's Gate Buildings, Dartmouth Street, London, S.W.1, returnable by December 20. (Quote U.119 CCE). Canvassing disqualifies.

(*Continued on next page.*)

MISCELLANEOUS ADVERTISEMENTS.

(*Continued from previous page.*)

COUNTY COUNCIL OF

THE WEST RIDING OF

YORKSHIRE

TO CONSTRUCTIONAL ENGINEERS
AND
REINFORCED CONCRETE CONTRACTORS.

CONSTRUCTIONAL ENGINEERS AND REINFORCED CONCRETE CONTRACTORS desirous of tendering for the provision and fixing of structural steelwork and reinforced concrete in building works for the County Council should make application for their names to be included on the County Council's LISTS OF SELECTED CONTRACTORS.

Contractors whose names are already upon the County Council's lists of selected contractors need not renew their application.

Projects are classified into the under-mentioned categories, and applicants for inclusion on the lists should state the category or categories of work for which they wish to be considered.

(*a*) Works of a value between £1,500 and £5,000
(*b*) Works of a value between £5,000 and £10,000
(*c*) Works of a value exceeding £10,000.

All applications should give particulars of works of a similar nature which have been carried out by the applicant, together with the names and addresses of three Architects or Engineers to whom reference can be made, and the name and address of the applicant's Banker.

Applications to be sent in writing to the Deputy County Architect, Bishopgarth, Westfield Road, Wakefield, not later than 10.0 a.m. on January 19, 1957.

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Clerk of the County Council

County Hall, Wakefield.

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Applications to:

W. F. C. Crozier, M.Sc., M.I.C.E.
27 Hammersmith Grove, London, W.6.

SITUATION VACANT. Engineer. Recent graduate in civil or mechanical engineering or practical engineer with plant experience required by rapidly-expanding international company. Initial appointment in London area. Interesting and varied work, and good opportunities. Apply to Ready Mixed Concrete, Ltd., 19 Dartmouth Street, London, S.W.1.

SITUATIONS WANTED.

SITUATION WANTED. Reinforced concrete engineer, with 22 years' design and site experience, requires a responsible position with consulting engineers or contractors. Box 4401, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

PART-TIME WORK. Detailing draughtsman, with seven years' experience, is ready to undertake work in evenings and at week-ends for consulting engineers. Any structure. Neat and accurate. Good lettering. Box 4402, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

PATENT

PATENTS. The Proprietors of British Patents Nos. 647,972 for "Improvements in Ceramic Constructional Units, and in Reinforced Beams and Floors Constructed Therefrom", 647,975 for "Improvements in the Construction of Floors by Means of Terra Cotta or like Blocks", and 663,910 for "Improvements in Hollow Moulded Blocks for Constructional Purposes", desire to enter into negotiations with a firm or firms for the sale of the patents or for the grant of licences thereunder. Further particulars may be obtained from Marks & Clerk, 57 and 58 Lincoln's Inn Fields, London, W.C.2.

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